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DESIGN OF STEEL STRUCTURES

By Limit State Method as Per IS 800-2007

FOURTH EDITION

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1

INTRODUCTION

Design of a building consists of two parts viz. (i) functional design and (ii) structural design. The first part consists in planning the building to serve its requirements taking into account ventilation, lighting, aesthetic view etc. The structural design consists in proportioning various elements of the building such that loads acting on it are transferred safely to the ground and at the same time unnecessarily excess material is not used.

For transferring the loads to the ground various materials, like asbestos sheets, tiles, bricks, cement concrete, reinforced concrete, steel, aluminium are used. However, main body of the present-day structures consists of R.C.C or steel. In tall structures composite construction of steel and concrete is also commonly used.

1.1 COMMON STEEL STRUCTURES

Steel has high strength per unit mass. Hence it is used in constructing large column-free structures. The following are the common steel structures in use:

1. Roof trusses for factories, cinema halls, auditoriums etc.
2. Trussed bents, crane girders, columns etc., in industrial structures.
3. Roof trusses and columns to cover platforms in railway stations and bus stands.
4. Single layer or double layer domes for auditoriums, exhibition halls, indoor stadiums etc.
5. Plate girder and truss bridges for railways and roads.
6. Transmission towers for microwave and electric power.
7. Water tanks.
8. Chimneys etc.

1.2 ADVANTAGES AND DISADVANTAGES OF STEEL STRUCTURES

The advantages of steel over other materials for construction are:

1. It has high strength per unit mass. Hence even for large structures, the size of steel structural element is small, saving space in construction and improving aesthetic view.

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2. It has assured quality and high durability.
3. Speed of construction is another important advantage of steel structure. Since standard sections of steel are available which can be prefabricated in the workshop/site, they may be kept ready by the time the site is ready and the structure erected as soon as the site is ready. Hence there is lot of saving in construction time.
4. Steel structures can be strengthened at any later time, if necessary. It needs just welding additional sections.
5. By using bolted connections, steel structures can be easily dismantled and transported to other sites quickly.
6. If joints are taken care, it is the best water and gas resistant structure. Hence can be used for making water tanks also.
7. Material is reusable.

The disadvantages of steel structures are:

1. It is susceptible to corrosion.
2. Maintenance cost is high, since it needs painting to prevent corrosion.
3. Steel members are costly.

1.3 TYPES OF STEEL

Steel is an alloy of iron and carbon. Apart from carbon by adding small percentage of manganese, sulphur, phosphorus, chrome nickel and copper special properties can be imparted to iron and a variety of steels can be produced. The effect of different chemical constituents on steel are generally as follows:

(i) Increased quantity of carbon and manganese imparts higher tensile strength and yields properties but lower ductility, which is more difficult to weld.
(ii) Increased sulphur and phosphorus beyond 0.06 percent imparts brittleness, affects weldability and fatigue strength.
(iii) Chrome and nickel impart corrosion resistance property to steel. It improves resistance to high temperature also.
(iv) Addition of a small quantity of copper also increases the resistance to corrosion.

By slightly varying chemical composition various types of steels are manufactured to be used as structural member, tubes, pipes, sheets, strips, reinforcements for R.C.C, rivets, bolts, nuts and for welding.

In this chapter mainly structural steels are discussed and their properties presented. The structural steel is the steel used for the manufacture of rolled steel sections. These rolled steel sections are used to form steel frameworks required in the structures.

Structural steel may be mainly classified as mild steel and high tensile steel.

Structural steel is also known as standard quality steel. Its requirements have been specified in IS 226-1975. This steel is also available in copper bearing quality in which case it is designated as Fe 410-Cu-S, where 410 refers to ultimate tensile strength of 410 Mpa (= 410 N/mm^2).

This is also known as grade E250 steel in which 250 refers to 250 Mpa yield strength. E300 (Fe-440) and E-350 (Fe 490) steels are also manufactured.

In high tensile steel mechanical properties and resistance to corrosion are enhanced by alloying with small proportions of some other alloys or increasing the carbon content. Standards of high tensile steel are covered in IS 961-1975. Weldable quality steels which are recommended by IS 2007 are designated as E410 (Fe 540), E450 (Fe 570)D and E450 (Fe 590)E. As per IS 800-2007, the structural steel used in general construction, coming under the purview shall conform to IS 2062 i.e., to weldable quality steel.

Structural steel other than those specified under mild steel and high tensile steel conforming to weldable quality may also be used provided that the permissible stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.

Steel (ordinary quality) that is not supported by mill test result may be permitted to be used for unimportant members, where their properties such as ductility and weldability do not affect the performance requirements of the structure as a whole.

In this book mild steel (structural steel-standard quality) and high tensile steel of weldable quality (conforming to IS 2062) are considered for the design.

1.4 PROPERTIES OF STRUCTURAL STEEL

The properties of steel required for engineering design may be classified as

(i) Physical Properties
(ii) Mechanical Properties.

(i) *Physical Properties*: Irrespective of its grade physical properties of steel may be taken as given below (clause 2.2.4 of IS 800-2007):

(a) Unit mass of steel, $\rho = 7850$ kg/m^3.
(b) Modulus of elasticity, $E = 2.0 \times 10^5$ N/mm^2.
(c) Poisson's ratio, $\mu = 0.3$.
(d) Modulus of rigidity, $G = 0.769 \times 10^5$ N/mm^2.
(e) Coefficient of thermal expansion, $\alpha_t = 12 \times 10^{-6}$/°C.

(ii) *Mechanical Properties:* The following are the important mechanical properties in the design:

(a) Yield stress f_y.
(b) The tensile or ultimate stress f_u.

(c) The maximum percentage elongation on a standard gauge length and
(d) Notch toughness.

Except for notch toughness, the other properties are determined by conducting tensile tests on samples cut from the plates, sections etc. IS 800-2007 gives mechanical properties of different types of structural steel products in its Table 1.1. Table 1.1 gives mechanical properties of structural steel conforming to IS 2062, which are used for the design in this book.

Table 1.1 Mechanical properties of structural steel conforming to IS Code 2062

Sl. No.	Grade/ Classification	Yield Stress in N/mm²			Ultimate Tensile Stress in N/mm²	Percentage Elongation
		t < 20 mm	t = 20–40 mm	t > 40 mm		
1	E250 (Fe 410W)A	250	240	230	410	23
2	E250 (Fe 410W)B	250	240	230	410	23
3	E250 (Fe 410W)C	250	240	230	410	23
4	E300 (Fe 440)	300	290	280	440	22
5	E350 (Fe 490)	350	330	320	490	22
6	E410 (Fe 540)	410	390	380	540	20
7	E450 (Fe 570)D	450	430	420	570	20
8	E450 (Fe 590)E	450	430	420	590	20

Notes:
1. Percentage elongation shall be taken over the gauge length 5.65 $\sqrt{S_o}$ where, S_o is original cross-sectional area of the specimen.
2. If elongation is non-proportional, 0.2 percent proof stress is taken as yield stress.

1.5 ROLLED STEEL SECTIONS

Like concrete, steel section of any shape and size cannot be cast on site, since steel needs very high temperature to melt it and roll into required shape. Steel sections of standard shapes, sizes and length are rolled in steel mills and marketed. User has to cut them to the required length and use required sections for the steel framework. Many steel sections are readily available in the market and are in frequent demand such steel sections are known as *Regular Steel Sections*. Some steel sections are not in use commonly, but the steel mills can roll them if orders are placed. Such steel sections are known as *Special Sections*.

Various types of rolled steel sections manufactured are listed below:

(i) Rolled steel I-sections (Beam sections)
(ii) Rolled steel Channel sections
(iii) Rolled steel Angle sections
(iv) Rolled steel Tee sections
(v) Rolled steel Bars
(vi) Rolled steel Tubes

(vii) Rolled steel Plates
(viii) Rolled steel Flats
(ix) Rolled steel Sheets and Strips.

Steel tables give nominal dimensions, weight per metre length and geometric properties of various rolled steel sections.

1.5.1 Rolled Steel I-section

The following five series of rolled steel I-sections are manufactured in India:

(a) Indian Standard Junior beams – ISJB
(b) Indian Standard Light Beams – ISLB
(c) Indian Standard Medium Beams – ISMB
(d) Indian Standard Wide-flange Beams – ISWB
(e) Indian Standard Heavy Beams – ISHB.

Figure 1.1 shows a typical I-section beam.

These sections are designated by the series to which they belong, followed by depth (in mm) and weight per metre run e.g. ISMB 500 @ 0.852 kN/m. It may not matter much if weight per metre length is not written in case of ISJB, ISLB and ISMB sections, since there is only one standard section for a specified depth. But in case of ISWB and ISHB sections weight per unit length should always be specified since for the same depth in these series more than one sections are available with different weight and properties e.g.

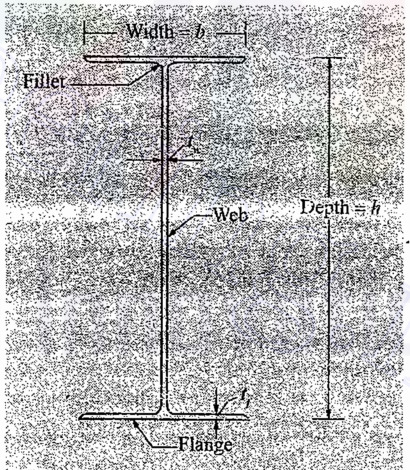


Figure 1.1 Rolled steel I-section.

ISWB 600 @ 1.423 kN/m
ISWB 600 @ 1.312 kN/m
ISHB 450 @ 0.855 kN/m
ISHB 450 @ 0.907 kN/m.

1.5.2 Rolled Steel Channel Sections

These sections are classified into the following four series:

(a) Indian Standard Junior Channel – ISJC
(b) Indian Standard Light Channel – ISLC
(c) Indian Standard Medium weight Channel – ISMC
(d) Indian Standard Special Channel – ISSC.

Figure 1.2 shows a typical channel section.

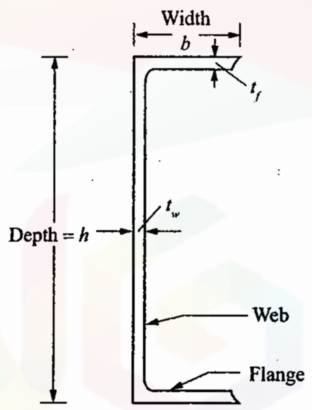


Figure 1.2 Rolled steel channel section.

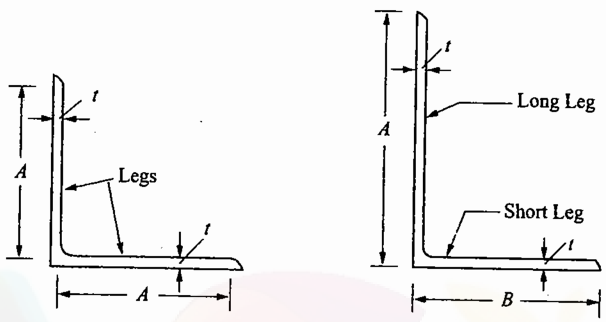
Rolled steel channel sections are designated by the series to which they belong, followed by depth (in mm) and weight (in kN/m). e.g. ISMC 300 @ 0.351 kN/m.

1.5.3 Rolled Steel Angle Sections

These are classified into the following two series:

(a) Indian Standard Equal Angle – ISA
(b) Indian Standard Unequal Angle – ISA.

Figure 1.3 shows typical sections.



(a) Rolled steel equal angle. (b) Rolled steel unequal angle.

Figure 1.3

Thickness of legs of equal and unequal angles are same. Rolled steel equal and unequal angles are designated by their series name ISA followed by length and thickness of legs e.g:

ISA 150 150, 12 mm thick or ISA 150 × 150 × 12
ISA 150 115, 10 mm thick or ISA 150 × 115 × 10.

1.5.4 Rolled Steel Tee Sections

Following five series of rolled steel sections are available:

(a) Indian Standard Normal Tee bars – ISNT
(b) Indian Standard Heavy flanged Tee bars – ISHT
(c) Indian Standard Special Legged Tee bars – ISSLT
(d) Indian Standard Light Tee bars – ISLT
(e) Indian Standard Junior Tee bars – ISJT.

A typical Tee section is shown in Fig. 1.4.

These rolled steel sections are designated by the series to which they belong followed by depth and weight per metre length. e.g:

ISNT 60 @ 53 N/m

As per IS 808-1984, the following T-sections have also been adopted:

(a) Indian Standard Deep legged Tee bars – ISDT
(b) Indian Standard slit Medium weight Tee bars – ISMT
(c) Indian Standard slit Heavy Tee bars from I-sections – ISHT.

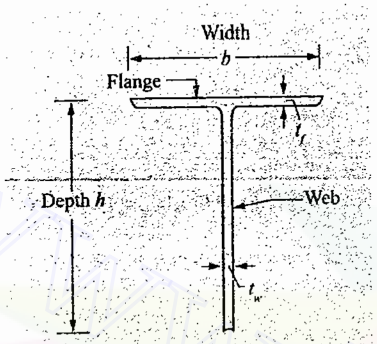


Figure 1.4 Rolled steel T-section.

1.5.5 Rolled Steel Bars

Rolled steel bars are classified into the following two series:

(a) Indian Standard Round bars – ISRO
(b) Indian Standard Square bars – ISSQ.

Rolled steel bars are designated by ISRO followed by diameter in case of round bars and ISSQ followed by side width in case of square bars e.g:

ISRO 16
ISRQ 20.

1.5.6 Rolled Steel Tubes

These sections are designated by their nominal bore sizes. In each size there are three classes, namely Light, Medium and Heavy. The difference is due to difference in their thicknesses. Hence their cross sectional properties are also different. For example, a 40 mm tube has 3 types and their sectional properties are as given below:

Nominal Bore	Outer Diameter	Class	Thickness in mm	Weight per m Length	Area mm^2	MI mm^4	K mm
		Light	2.90	31.9 N	414	107×10^3	16.1
40	48.3	Medium	3.25	35.4 N	460	117.3×10^3	16.0
		Heavy	4.05	435 N	563	139×10^3	15.7

1.5.7 Rolled Steel Plates

Rolled steel plates of the following thicknesses are available:
5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56, 63, 71, 80 mm.

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They are rolled in the widths
160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000, 1100, 1250, 1400, 1600, 1800, 2000, 2200, 2500 mm.

These plates are designated by ISPL followed by length, width and thickness. e.g:
ISPL 2000 × 1000 × 6.

1.5.8 Rolled Steel Strips

Rolled steel strip is designated as ISST followed by width and thickness. These sections are available in the following width and thickness:

Width: 100, 110, 125, 140, 160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000 mm.

Thickness: 0.8, 0.9, 1.0, 1.1, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.5, 2.8, 3.2, 3.5, 4.0, 4.5 mm.

It may be noted that thickness of strips is less than 5 mm. Rolled steel strip is designated as ISST, followed by width and thickness e.g:

ISST 250 × 2.5 mm.

1.5.9 Rolled Steel Flats

Flats differ from strips in the sense that the thickness of flats is 5 mm onward and their width is limited. Flats of the following width and thickness are listed in IS Handbook.

Width: 12, 16, 20, 25, 32, 40, 50, 63, 80, 100, 125, 160, 200, 250 mm.

Thickness: 5, 5.5, 6, 7, 8, 9, 10, 11, 12, 14, 16, 18, 20, 22, 25 mm.

They are designated by width followed by letters ISF and thickness e.g:

80 ISF 10 means, 80 mm wide Indian Standard Flat of thickness 10 mm.

1.6 SPECIAL CONSIDERATIONS IN STEEL DESIGN

The following special considerations are required in the steel design:

1. Size and Shape
2. Buckling
3. Minimum Thickness
4. Connection Designs.

1. *Size and Shape:* Steel is manufactured in steel mills and is available in certain shapes and sizes. Hence the member of a steel structure should be designed to consist of any of the available sections or a combination of them. For example, a beam section may be a standard I-section or it may consist of built up sections as shown in Fig. 1.5.

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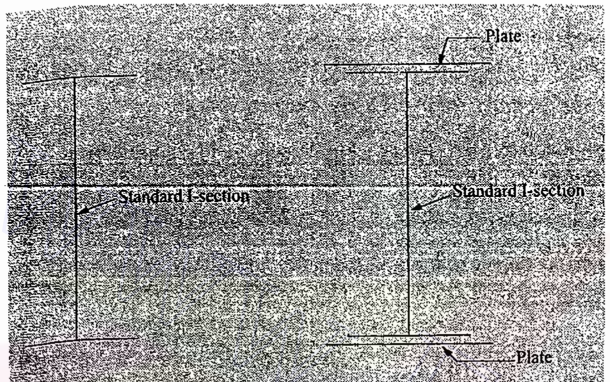

Figure 1.5 Standard section and built up section.

Sometimes the choice of the section of a member is governed by the shape of the other member and the type of the joint between the two members.

2. *Buckling Consideration:* The permissible load per unit area in steel is much higher as compared to permissible values in concrete. Therefore, for the same load, the cross sectional area of a steel member is smaller. As the members in a steel structure are more slender, the compression members in steel structure are liable to buckling. In case of beams, there are chances of lateral buckling which creates special problems. As a steel member consists of a number of thin plates, the stability of each part is to be considered. To account for buckling phenomenon, codes specify that part of sections be taken as ineffective.

3. *Minimum Thickness:* Corrosion needs special consideration in steel design. If very thin sections are used, a small amount of corrosion may result into a large percentage reduction in effective area. Hence design practice specify minimum thicknesses to be used in structural members. For the members directly exposed to weather the following minimum thickness is to be used:

(a) If fully accessible for cleaning and painting – 6 mm.
(b) If not accessible for cleaning and painting – 8 mm.
(c) The above limitations do not apply for rolled steel sections, tubes and cold formed light gauge sections. However IS 800-2007, has dropped the specification for minimum thickness.

4. *Need for Design of Connections:* A steel design is not complete if the following connections are not designed:

(a) Connections between various standard sections selected for a member
(b) Connections between various members (like beam, column, foundation etc.) of the structure.

The following three types of connections are commonly used:

(a) Riveted Connections
(b) Bolted Connections
(c) Welded Connections.

However now-a-days riveted connection is being given up. IS 800-2007 recommends use of welded connection. But bolted connections are required when various components are fabricated separately and are assembled at the site to get required structure.

1.7 LOADS

Various loads expected to act on a structure may be classified as given below:

(a) Dead Loads (DL)
(b) Imposed Loads (IL)
(c) Wind Loads (WL)
(d) Earthquake Loads (EL)
(e) Erection Loads (ER)
(f) Accidental Loads (AL)
(g) Secondary Effects.

(a) *Dead Loads:* Dead loads include the weight of all permanent construction. For example, in a building weight of roofs, floors, floor finishes, wall, beams, columns, footing, architectural finishing materials etc., constitute dead load. These loads may be assessed by estimating the quantity of each material and then multiplying it with unit weight. The unit weight of various materials in a structure are given in IS code 875 (part I). It gives a exhaustive list. For example, under the heading 'brick masonry' it has four types like common clay bricks, engineering bricks, glazed bricks and pressed bricks. Under the heading plain concrete there are 10 groups. The commonly used values are listed in Table 1.2.

It is to be noted that under dead load self weight of the structure being designed also should be included. Since, without design, the self weight is not known, this is estimated from reference to similar other

Table 1.2 Unit weight of important building materials

Sl. No.	Material	Unit Weight
1	Asbestos sheets	0.130 kN/m^2
2	Mangalore tile with battens	0.785 kN/m^2
3	Terrazo paving of floors	0.236 kN/m^2
4	Plain concrete	24 kN/m^3
5	Reinforced concrete	25 kN/m^3
6	Brick masonry	20 kN/m^3
7	Cement plaster	20 kN/m^3
8	Granite stone masonry	24 kN/m^3
9	Steel	78.5 kN/m^3

structures or by empirical formulae based on past experience. After completing the design it may be calculated and compared with estimated value. If the difference is substantial, redesign is required. For majority of structures, it is not found necessary to redesign.

(b) *Imposed Loads:* IS 800-2007 groups the following loads as imposed loads:

(i) Live load
(ii) Crane load
(iii) Snow load
(iv) Dust load
(v) Hydrostatic and earth pressure
(vi) Impact load
(vii) Horizontal loads on parapets and balustrades.

(i) LIVE LOADS (LL): The loads which keep on changing from time to time are called live loads. Common examples of such loads in a building are the weight of the persons, weight of movable partition, dust loads and weight of furniture. These loads are to be suitably assumed by the designer. It is one of the major loads in the design. The minimum values to be assumed are given in IS 875 (part 2)-1987. It depends upon the intended use of the building. These values are presented for square metre of floor area. The code gives the values of loads for the following occupancy classification:

(a) Residential buildings–dwelling houses, hotels, hostels, boiler rooms and plant rooms, garages
(b) Educational buildings
(c) Institutional buildings
(d) Assembly buildings
(e) Business and office buildings
(f) Mercantine buildings
(g) Industrial buildings, and
(h) Storage rooms.

The code gives uniformly distributed load as well as concentrated loads. The structures are to be investigated for both uniformly distributed and worst position of concentrated loads. The one which gives worst effect is to be considered for the design but both should not be considered to act simultaneously.

In a particular building, live load may change from room to room. For example, in a hotel or a hostel building the loads specified are,

	UDL	Concentrated Load
(a) Living rooms and bedrooms	2 kN/m^2	1.8 kN
(b) Kitchen	3 kN/m^2	4.5 kN
(c) Dining rooms	4 kN/m^2	2.7 kN
(d) Office rooms	2.5 kN/m^2	2.7 kN
(e) Store rooms	5 kN/m^2	4.5 kN
(f) Rooms for indoor games	3 kN/m^2	1.8 kN
(g) Bathrooms and toilets	2 kN/m^2	–
(h) Corridors, passages, staircases etc., and	3 kN/m^2	4.5 kN
(i) Balconies	4 kN/m^2	1.5 kN concentrated at outer edge

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Table 1.3 Minimum live load to be considered

Sl. No.	Occupancy	UDL Load	Concentrated
1	Bathrooms and toilets in all types of building	2 kN/m^2	1.8 kN
2	Living and bedrooms	2 kN/m^2	1.8 kN
3	*Office rooms in*		
	(i) Hostels, hotels, hospitals and business building with separate store	2.5 kN/m^2	2.7 kN
	(ii) In assembly buildings	3 kN/m^2	4.5 kN
4	*Kitchens in*		
	(i) Dwelling houses	2 kN/m^2	1.8 kN
	(ii) Hostels, hotels and hospitals	3 kN/m^2	4.5 kN
5	Banking halls, classrooms, X-ray rooms, operation rooms	3 kN/m^2	4.5 kN
6	*Dining rooms in*		
	(i) Educational buildings, institutional and mercantile buildings	3 kN/m^2	2.7 kN
	(ii) Hostels and hotels	4 kN/m^2	2.7 kN
7	*Corridors, passages, staircases in*		
	(i) Dwelling houses, hostels and hotels	3 kN/m^2	4.5 kN
	(ii) Educational institutional and assembly buildings	4 kN/m^2	4.5 kN
	(iii) Mercantine buildings	5 kN/m^2	4.5 kN
8	*Reading rooms in libraries*		
	(i) With separate storage	3 kN/m^2	4.5 kN
	(ii) Without separate storage	4 kN/m^2	4.5 kN
9	*Assembly areas in assembly buildings*		
	(i) With fixed seats	5 kN/m^2	–
	(ii) Without fixed seats	5 kN/m^2	3.6 kN
10	Store rooms in educational buildings	5 kN/m^2	4.5 kN
11	Store room in libraries	6 kN/m^2 for a height of 2.24 + 2 kN/m^2 for every 1 m additional height	4.5 kN
12	*Boiler rooms and plant rooms in*		
	(i) Hostels, hotels, hospitals, mercantile and industrial buildings	5 kN/m^2	4.5 kN
	(ii) Assembly and storage buildings	7.5 kN/m^2	4.5 kN

Some of the important values are presented in Table 1.3 which are the minimum values and wherever necessary more than these values are to be assumed.

Live loads to be considered on various roofs are presented in Table 1.4.

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Table 1.4 Live loads on various types of roofs

Sl. No.	Type of Roof	Live Load Measured on Plan Area	Minimum Live Load Measured on Plan
(i)	Flat, sloping or curved roof with slopes up to and including 10 degrees		
	(a) Access provided	1.5 kN/m^2	3.75 kN uniformly distributed over any span of one metre width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall.
	(b) Access not provided except for maintenance	0.75 kN/m^2	1.9 kN uniformly distributed over any span of one metre width of the roof slab and 4.5 kN uniformly distributed over the span of any beam of truss or wall.
(ii)	Sloping roof with slope greater than 10 degrees	For roof membrane sheets or purlins – 0.75 kN/m^2 less 0.02 kN/m^2 for every degree increase in slope over 10 degrees	Subject to a minimum of 0.4 kN/m^2
(iii)	Curved roof with slope of line obtained by joining springing point to the crown with the horizontal, greater than 10 degrees	$(0.75 - 0.52\,\alpha^2)$ kN/m^2 where $\alpha = \frac{h}{l}$; h = height of the highest point of the structure measured from its springing and; l = chord width of the roof if singly curved and shorter of the two sides if doubly curved. Alternatively, where structural analysis can be carried out for curved roofs of all slopes in a simple manner applying the laws of statistics, the curved roofs shall be divided into minimum 6 equal segments and for each segment imposed load shall be calculated appropriate of each segment as given in (i) and (ii)	Subject to a minimum of 0.4 kN/m^2

Note: 1 The loads given above do not include loads due to snow, rain, dust collection, etc. The roof shall be designed for live loads given above or snow/rain load, whichever is greater.

Note: 2 For special types of roofs with highly permeable and absorbent material, the contingency of roof material increasing in weight due to absorption of moisture shall be provided for.

However in multi-storeyed buildings chances of full imposed loads acting simultaneously on all floors is very rare. Hence the code makes provision for reduction of loads in designing columns, load bearing walls, their supports and foundations as shown in Table 1.5.

Table 1.5 Reduction in live loads on floors in design of supporting structural elements

Number of Floors (including the roof) to be carried by Member Under Consideration	Reduction in Total Distributed Live Load in Percent
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

(ii) Crane Loads (CL): These loads include loads from cranes and other machines acting on the structure. The loads may be taken as per manufacturers/suppliers data. In the absence of specific indications they may be taken as given below (IS 800-2007, clause 3.5.4).

(a) Vertical loads + full impact from one crane or two cranes in case of one behind another operation plus vertical load of other cranes in the bay.
(b) Horizontal thrust one crane only or two in case of one behind another operation in the bay.
(c) In case of multibay multicrane gantries only, two bays of building cross sections may be considered.
(d) The longitudinal thrust on a crane track rail shall be considered for a maximum of two loaded cranes on the track.
(e) Lateral thrust and longitudinal thrust acting across and along the crane rail respectively, shall be assumed not to act simultaneously. The effect of each force, shall however be investigated separately.

(iii) Snow Load: IS 875 (part 4) deals with snow loads on roof of the buildings. This load is to be considered for the buildings to be located in the regions where snow is likely to fall. The snow load acts vertically downward.

It may be expressed in kN/m^2. The load on the roof due to accumulation of snow is obtained by the expression

$$S = \mu S_0$$

where

S = Snow load on plan area of roof,
μ = Shape coefficient, and
S_0 = Ground snow load.

Ground snow load at any place depends on the critical combination of the maximum depth of undisturbed aggregate cumulative snowfall and its average density. These values for different regions may be obtained from Snow and Avalanches Study Establishment, Manali (H.P) or from Indian Meteorological Department, Pune (Maharashtra). The shape coefficient depends upon the shape of roof. IS 875 (part 4) gives these values for some of the common shapes. When the slope of the roof is more than 60° this load is not to be considered.

It may be noted that roofs should be designed for the actual load due to snow or for the live load, whichever is more severe.

IS 875 (part 4) gives ice load on wires also. Such loads are to be considered in the design of overhead electrical transmission towers in the zones subjected to snowfall. The thickness of ice deposit around wire may be taken as between 3 and 10 mm depending upon the location of the structure. The mass density of ice may be assumed as 0.9 g/cm^3 (9 kN/m^3). While considering the wind force on wires and cables, the increase in diameter due to ice deposit shall be taken into consideration.

(iv) Dust Load: In areas prone to settlement of dust on roof (e.g: steel plants, cement plants) provision for dust load equivalent to probable thickness of accumulation of dust may be made.

(v) Hydrostatic and Earth Pressure: IS 875 (part 5) gives specifications for considering such loads. In the design of structures partly or fully below ground level, the pressure exerted by soil or water or both shall be duly accounted on the basis of established theories. All foundation slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift equal to the full uplift hydrostatic pressure.

(vi) Impact Load: For structures supporting moving loads suitable additional allowance of load should be made by increasing imposed load. For example:

Structure	Impact Allowance, Percent, Minimum
(a) For frames supporting lifts and hoists	100
(b) For foundations, footings and piers supporting lifts and hoisting apparatus	40
(c) For supporting structures and foundations for light machinery, shafts or motor units	20
(d) For supporting structures and foundations for reciprocating machinery or power unit	50
(e) For girders supporting electric wire near head cranes	25
(f) For girders supporting hand operated cranes	10

Table 1.6 Horizontal loads on parapets and balustrades

Sl. No.	Usage Area	kN/m
1	Light access stairs, not more than 600 mm wide	0.25
2	Light access stairs, more than 600 mm wide	0.35
3	All other stairways (except those subject to overcrowding)	0.75
4	In place of assembly such as theatres, schools, auditoriums, stadiums and buildings likely to be overcrowded	2.25

(vii) Horizontal Loads: Parapets, balustrades and their supporting structures shall be designed for the horizontal forces acting at the hand rail or coping level. These loads may be considered to act vertically also but not simultaneously with the horizontal forces. The values given in Table 1.6 are minimum values and where values for actual loadings are available shall be used.

(c) *Wind Loads:* The force exerted by the horizontal component of wind is to be considered in the design of buildings, towers etc. The wind force depends upon the velocity of wind, shape, size and location of building. Complete details of calculating wind load on structure is given in IS 875 (part 3). Brief idea of these provisions is given below:

(i) Using colour code, the basic wind pressure 'V_b' is shown in a map of India. Designer can pickup the value of V_b depending upon the location of the structure.
(ii) To get the design wind velocity V_z, the following expression shall be used:

$$V_z = k_1\, k_2\, k_3\, V_b$$

where k_1 = Risk coefficient
k_2 = Coefficient based on terrain, height and structure size
k_3 = Topography factor

(iii) The design wind pressure is given by

$$P_z = 0.6\, V_z^2$$

where P_z is in N/m^2 at height h and V_z in m/sec. Upto a height of 30 m, the wind pressure is considered to act uniformly. Above 30 m height, the wind pressure increases.

(d) *Earthquake Loads:* Earthquake shocks cause movement of foundation of structures. Due to inertia additional forces develop on superstructure. The total vibration caused by earthquake may be resolved into three mutually perpendicular directions, usually taken as vertical and two horizontal directions. The movement in vertical direction do not cause significant forces in superstructure. But movement in horizontal direction needs special consideration.

The intensity of vibration of ground expected at any location depends upon the magnitude of earthquake, depth of focus, distance from the epicentre and the strata on which the structure stands.

The response of the structure to the ground vibration is a function of the foundation, soil, size and mode of construction and the duration and intensity of ground motion. IS 1893 gives the details of such calculations for structures standing on soils which will not settle considerably or slide appreciably due to earthquake. The seismic accelerations for the design may be arrived from seismic coefficients, which is defined as the ratio of acceleration due to earthquake and acceleration due to gravity. For the purpose of determining seismic forces, India is divided into five zones.

One of the following two methods may be used for computing seismic forces:

(i) Seismic coefficient method.
(ii) Response spectrum method.

The details of these methods are presented in IS:1893 and also in National Building Code of India. After Gujarat earthquake (2000), Government of India has realized the importance of structural design based on considering seismic forces and has initiated training of the teachers of technical institution on a large scale (NPEEE).

There are large number of cases of less importance and relatively small structures for which no analysis be made for earthquake forces provided certain simple precautions are taken in the construction. For example

(i) Providing bracings in vertical panels of steel and R.C.C. frames.
(ii) Avoiding mud and rubble masonry and going for light materials.

(e) *Erection Loads:* Prefabricated or precast members are subjected to different types of supports and different types of loads during erection compared to the types of supports and types of loads after erection. It is the responsibility of engineer to see that the structure or part of the structure do not fail during erection. Many cases of such failures are reported and in all cases engineers are held responsible. During erection storage of materials, equipment and impact of hoisting equipment cause special loads. Dead load, wind load and imposed live load during erections shall be considered along with the special erection loads. Special provisions shall be made including temporary bracing to take care of all such loads during erection.

(f) *Accidental Loads:* IS 875 (part 5) gives certain guidelines to take care of the following accidental loads on the structures:

(i) Impact and collision
(ii) Explosions and
(iii) Fire.

The probability of occurrence of such loads may be quite less but if it occurs the consequences are severe.

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(i) IMPACT AND COLLISIONS: Common sources of impacts are

- Vehicles
- Dropped object from cranes, lifts etc.
- Crane / lift failures
- Flying fragments.

IS 875 (part 5) gives requirement for estimating impact from collision of vehicles with structures. It is to be assumed that the vehicle strikes the structural elements at a height of 1.2 m in any possible direction and at a speed of 36 kmph. The fictitious vehicle shall be considered to consist of two masses $m_1 = 400$ kg and $m_2 = 12000$ kg which during compression of the vehicle produce an impact force increasing from zero, corresponding to the rigidities $c_1 = 10{,}000$ **kN/m** to $c_2 = 300$ **kN/m**.

Safety railings may be designed to protect the structures against such collisions.

(ii) EXPLOSIONS: The following types of explosions are to be looked into the designs:

- Internal gas explosion
- External gas explosion
- Boiler failures
- High explosions (dynamite).

IS 875 (part 5) gives codal requirements for internal gas explosions. The last type of explosion given above is to be considered in designing air raid shelters.

(iii) FIRE: IS 875 (part 5) specify that extraordinary loads during fire on escape routes and loads on another structure from structure failing during fire should be considered. To find thermal effect during fire any one of the following methods may be used:

- Time temperature curve and required fire resistance (minutes) or
- Energy balance method.

(g) *Secondary Effects:* The following types of secondary effects should be looked into the design:

- Differential settlement of foundations
- Differential shortening of columns
- Eccentric connections
- Rigidity of joints differing from design assumptions.

Thus a designer has to look into various loads likely to act during the life of the structure. He has to look into the following codes:

1. IS 875: Code of practice for structural safety of buildings.
2. IS 1893: Criteria for earthquake resistant design of structures.
3. IRC – Standard specifications and code of practice for road bridges.

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4. Railway bridge rules framed by railway design and standards organisations.
5. IS 4991: Criteria for blast resistant design of structures for explosions above ground.

1.8 LOAD COMBINATIONS

A judicious combination of the loads is necessary to ensure the required safety and economy in the design keeping in view the probability of

(a) their acting together
(b) their disposition in relation to other loads and severity of stresses or deformation caused by the combination of various loads.

The recommended load combinations by IS 875 are as given below.

1	DL	7	DL + IL + EL
2	DL + IL	8	DL + IL + TL
3	DL + WL	9	DL + WL + TL
4	DL + EL	10	DL + EL + TL
5	DL + TL	11	DL + IL + WL + TL
6	DL + IL + WL	12	DL + IL + EL + TL

Where

DL = Dead Load   IL = Imposed Load
WL = Wind Load   EL = Earthquake Load
and TL = Temperature Load.

Note: When snow load is present on roofs, replace imposed load by snow load for the purpose of above load combinations.

1.9 STRUCTURAL ANALYSIS

Structural analysis is necessary to find the internal forces developed in the members of the structures. The required internal forces for design are axial forces and moments. IS code permits the following methods of analysis:

(a) Elastic Analysis
(b) Plastic Analysis
(c) Advanced Analysis
(d) Dynamic Analysis.

(a) *Elastic Analysis:* It is based on the assumption that no fibre of the member has yielded for the design load and stress is linearly proportional to strain. The analysis may be in two stages.

Stage 1–First Order Analysis: It is based on the loads acting on undeformed geometry of the structure. Redistribution of 15% of peak moment is permitted by code.

Stage 2–Second Order Analysis: It is based on the deformed (large deflection theory) shape of the structure. IS 800 permits use of amplification factors (as given under clause 4.4.3.2) instead of second order analysis.

(b) *Plastic Analysis:* In this method it is assumed that when every fibre at a section reaches yield stress a plastic hinge is formed. After hinge is formed, it is assumed that the member rotates freely at the plastic hinge without resisting any additional moment. However its resistance to moment remains constant (M_p). This is called first order plastic analysis.

Code permits second order in elastic analysis by any of the following methods:

(i) Distributed plasticity method
(ii) Elastic – plastic hinge method
(iii) Modified plastic hinge method.

(c) *Advanced Analysis:* For a frame with full lateral restraints, an advanced structural analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frames. The analysis shall take into account the following:

(i) Relevant material properties
(ii) Residual stresses
(iii) Geometric imperfections
(iv) Reduction in stiffness due to axial compressions
(v) Second order effects
(vi) Erection procedure
(vii) Interaction with foundation.

For further details Annexure B of IS 800-2007 may be referred.

(d) *Dynamic Analysis:* Dynamic analysis is to be carried out in accordance with IS 1893 (part I).

1.10 DESIGN PHILOSOPHY

The aim of design is to decide shape, size and connection details of the members so that the structure being designed will perform satisfactorily during its intended life. With an appropriate degree of safety the structure should

(a) Sustain all loads expected on it.
(b) Sustain deformations during and after construction.
(c) Should have adequate durability.

(d) Should have adequate resistance to misuse and fire.
(e) Structure should be stable and have alternate load paths to prevent overall collapse under accidental loading. Analytical method of design consists in idealizing the structure, quantifying expected loads, carrying analysis to find member forces and sizing the members based on possible failure criteria. Since there are limitations in precisely modelling the structure, working condition is kept as a fraction of failure condition. The design philosophies used are listed below in the order of their evolution and they are briefly explained:

(i) Working Stress Method (WSM)
(ii) Ultimate Load Design (ULD) and
(iii) Limit State Design (LSD).

(i) WORKING STRESS METHOD: This is the oldest systematic analytical design method. Though IS 800-2007 insists for the limit state design, permits use of this method wherever LSD cannot be conveniently adopted.

In this method stress strain relation is considered linear till the yield stress. To take care of uncertainties in the design, permissible stress is kept as a fraction of yield stress, the ratio of yield stress to working stress itself known as factor of safety. The members are sized so as to keep the stresses within the permissible value. Thus

$$\text{permissible stress} = \frac{\text{yield stress}}{\text{factor of safety}}$$

The following load combinations are considered and increase of permissible stress by 33% is permitted when DL, LL and WL are considered:

Stress due to DL + LL ≤ permissible stress
Stress due to DL + WL ≤ permissible stress
Stress due to DL + LL + WL ≤ 1.33 permissible stress.

The limitations of WSM

The limitations of working stress method are:

1. It gives the impression that factor of safety times the working load is the failure load, which is not true. Actually it is much more, because a material can resist the load after yield appears at a fibre. In the indeterminate structures just formation of a plastic hinge is not the failure criteria, since it can resist load till some more hinges are formed resulting into collapse mechanism. Thus the redistribution of moments gives rise to the additional load carrying capacity.
2. It gives uneconomical sections.

Advantages of WSM

1. This method is simple.
2. This is reasonably reliable.
3. As the working stresses are low, the serviceability requirements are satisfied automatically.

(ii) ULTIMATE LOAD METHOD: The limitation of working stress method to assess actual load carrying capacity, made researchers to develope ultimate load method, which is also known as *load factor method* (LFM). When applied to steel structure it is referred as *plastic design method*. In this method a section is said to have formed plastic hinge when all the fibres yield. After that it continues to resist load which has caused plastic hinge but will not resist any more load. But structure continues to resist further load till sufficient plastic hinges are formed to develope collapse mechanism.

In this method safety measures are introduced by suggesting a load factor, which is defined as the ratio of design load to working load. The suggested load factors as per IS 800:1984 were as shown in Table 1.7.

Table 1.7

Sl. No.	Working Load	Minimum Load Factor
1	Dead Load	1.7
2	Dead Load + Imposed Load	1.7
3	Dead Load + Wind or Seismic Load	1.7
4	Dead Load + Imposed Load + Wind or Seismic Load	1.3

Advantages of ULD

1. Redistribution of internal forces is accounted.
2. It allows varied selection of load factors.

Disadvantages of ULD

It does not guarantee serviceability performance. To account for this IS 800:1984 suggested limitations on deflection. However it did not guarantee other serviceability limits like instability and fatigues etc. Finally it was felt to suggest more comprehensive method to take care of design requirements from strength and serviceability criteria.

(iii) LIMIT STATE DESIGN: It is the comprehensive method which will take care of both strength and serviceability requirements. IS 800:2007 suggests use of this method widely and restrict working stress method only wherever LSD cannot be applied. This method is thoroughly explained in chapter 2.

Example 1.1

Find the design load for an interior column of ground floor of an eight-storey building for the following data:

(i) Height of each floor = 3.5 m
(ii) Spacing of columns c/c in each direction = 3.8 m
(iii) Live load on roof = 1.5 kN/m^2
(iv) Live load on each floor = 2.5 kN/m^2
(v) Thickness of R.C. slab = 120 mm
(vi) Dead weight of floor finish = 1 kN/m^2
(vii) Weight of wall and beam = 10.8 kN/m

How much is the design load on the column of 5th floor?

Solution:

(a) On column of first floor

Characteristic Loads:

Dead Load:

Weight of R.C. slab = $0.120 \times 1 \times 1 \times 25 = 3.0$ kN/m^2

Weight of floor finish = 1 kN/m^2

Total dead load on floor = 4 kN/m^2

It is reasonable to assume each interior column takes load from an area = $3.8 \times 3.8 = 14.44$ m^2

$\therefore$ DL from floor = $4 \times 14.44 = 57.76$ kN

Dead load of wall and beam:

It is reasonable to assume each interior column takes load from 3.8 + 3.8 = 7.6 m length of beam.

$\therefore$ DL of wall and self weight of beam = $10.8 \times 7.6 = 82.08$ kN

$\therefore$ Total dead load from each floor = $82.08 + 57.76$

= 139.84 kN

Including self weight of column, load from each floor is say 150 kN.

$\therefore$ Dead load on ground floor column of eight-storey building,

= $150 \times 8 = 1200$ kN.

Live load:

Live load from top storey = $1.5 \times 3.8 \times 3.8 = 21.66$ kN

Live loads from each of other floors = $2.5 \times 3.8 \times 3.8 = 36.1$ kN

Reductions are to be made in live load while calculating load on columns, since all floors may not be fully loaded at a time. For this reduction factors as presented in Table 1.5 are to be used. Figure 1.6 shows the percentage reductions to be applied to live loads from different floors while finding live load on ground floor column and Table 1.8 shows the characteristic loads to be considered.

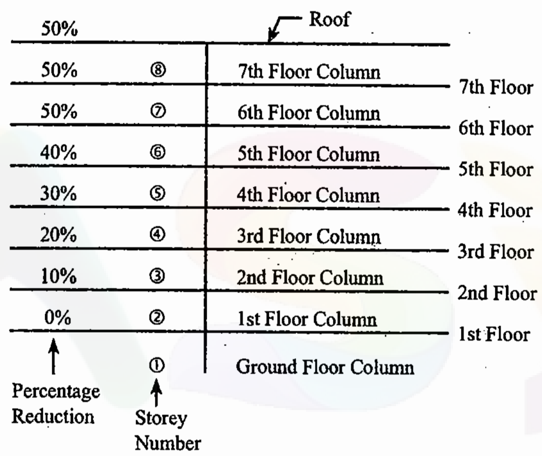

Figure 1.6 Percentage reduction for LL while finding set for ground floor column.

Table 1.8 Characteristic loads for LL

Column of Floor	LL Load in kN	Load to be Reduced in kN	Load to be Considered in kN
8th (Roof)	21.66	$0.50 \times 21.66 = 10.83$	10.83
7th	36.1	$0.50 \times 36.1 = 18.05$	18.05
6th	36.1	$0.50 \times 36.1 = 18.05$	18.05
5th	36.1	$0.40 \times 36.1 = 14.44$	21.66
4th	36.1	$0.30 \times 36.1 = 10.83$	25.27
3rd	36.1	$0.20 \times 36.1 = 7.22$	28.88
2nd	36.1	$0.10 \times 36.1 = 3.61$	32.49
1st	36.1	0	36.1
			$\Sigma = 191.33$ kN

Thus,

Characteristic dead load = 1200 kN

Characteristic live load = 191.33 kN

∴ Design Load = 1.5 DL + 1.5 WL

= 1.5 (1200 + 191.33)

= 2087.0 kN **Answer**

Load on column of 5th floor:

Dead Load = 150 kN from each floor above it.

There are 3 floors above it.

∴ Dead load in column of 5th floor = 150 × 3 = 450 kN.

Figure 1.7 shows percentage reduction to be considered for load on 5th floor.

% Reduction of LL	
20%	Roof (8th floor)
10%	7th Floor
0%	6th Floor
	5th Floor Column

Figure 1.7 Percentage reduction for LL for 5th floor load calculation.

LL from roof (8th floor) = 21.66 × (1 – 0.2) = 17.328 kN

LL from 7th floor = 36.1 × (1 – 0.1) = 32.490 kN

LL from 6th floor = 36.1 kN

∴ Total LL = 17.328 + 32.490 + 36.100

= 85.918 kN

∴ Design load on column of 5th floor = 1.5 × 450 + 1.5 × 85.918

= 803.877 kN **Answer**

* * *

Questions

1. Explain the advantages and disadvantages of using steel structures.
2. Explain what is structural steel. List out the important properties of such steel.
3. Explain the special considerations required in the design of steel structures.
4. Explain briefly various types of loads to be considered in design of steel structures.

2

PRINCIPLES OF LIMIT STATE DESIGN

Aim of a design is to see that the structure built is safe and it serves the purpose for which it is built. A structure may become unfit for use not only when it collapses but also when it violates the serviceability requirements of deflections, vibrations, cracks due to fatigue, corrosion and fire. In this method of design various limiting conditions are fixed to consider a structure as fit. At any stage of its designed life (120 years for permanent structures), the structure should not exceed these limiting conditions. The design is based on probable load and probable strength of materials. These are to be selected on probabilistic approach. The safety factor for each limiting condition may vary depending upon the risk involved. It is not necessary to design every structure to withstand exceptional events like blast and earthquake. In limit state design risk based evaluation criteria is included. Thus the philosophy of limit state design method is to see that the structure remains fit for use throughout its designed life by remaining within the acceptable limit of safety and serviceability requirements based on the risks involved.

2.1 DESIGN REQUIREMENTS

Steel structure designed and constructed should satisfy the requirements regarding stability, strength, serviceability, brittle fracture, fatigue, fire and durability. The structures should meet the following requirements (IS 800-2007, clause 5.1.2):

(A) Remain fit with adequate reliability and be able to sustain all loads and other influences experienced during construction and use.

(B) Have adequate durability under normal maintenance.

(C) Do not suffer overall damage or collapse disproportionately under accidental events like explosions, vehicle impact or due to consequences of human error to an extent beyond local damage. The catastrophic damage shall be limited or avoided by appropriate choice of one or more of the following:

(a) Avoiding, eliminating or reducing exposure to hazards, which the structure is likely to sustain.

(b) Choosing structural forms, layouts and details and designing such that:

(i) the structure has low sensitivity to hazardous conditions and

(ii) the structure survives with only local damage even after serious damage to any one individual element by the hazard.

(c) Choosing suitable material, designing and detailing procedure and construction procedure as relevant to the particular structure.

The collapse is considered disproportionate, if more than 15 percent of the floor or roof area of 70 m^2 collapses at that level and one adjoining level either above or below it, under a load equal to 1.05 or 0.9 times the dead load, 0.33 times temporary or full imposed load of permanent nature and 0.33 times wind load acting together. To avoid disproportionate collapse, the following conditions should be satisfied:

(a) The building should be effectively tied together at each principal floor level and each column should be effectively held in position. These ties may be steel members such as beams which may be designed for other purposes or the shear connectors which connect floor with beams and columns. These connections should be capable of resisting:
   (i) expected tensile force subjected to a minimum of 75 kN.
   (ii) one percent of the maximum axial compression in the column.

(b) All column splices should be capable of resisting a tensile force equal to the largest of a factored dead and live load from the floor above or below the splice.

(c) Lateral load system to resist horizontal loads should be distributed throughout the building in nearly orthogonal directions.

(d) Floor or roof units should be effectively anchored in the direction of their spans either to each other or directly to the support.

If above provisions are not made design should be checked for disproportionate collapse.

2.2 LIMIT STATES

Limit states are the states beyond which the structure no longer satisfied the specified performance requirements. The various limit states to be considered in design may be grouped into the following two major categories:

(a) Limit state of strength
(b) Limit state of serviceability.

(a) *Limit state of strength:*

The limit states, prescribed to avoid collapse of structure which may endanger the safety of life and property, are grouped under this category. The limit state of strength includes:

(i) Loss of equilibrium of whole or part of the structure.
(ii) Loss of stability of structure as a whole or part of it.
(iii) Failure by excessive deformation.

(iv) Fracture due to fatigue.
(v) Brittle fracture.

(b) *Limit state of serviceability:*

The limit state of serviceability include:

(i) Deformations and deflections adversely affecting the appearance or effective use of structure or causing improper functioning of equipment or services or causing damage to finishings.
(ii) Vibrations in structures or any part of its component limiting its functional effectiveness.
(iii) Repairable damage or crack due to fatigue.
(iv) Corrosion.
(v) Fire.

2.3 ACTIONS (LOADS)

The actions to be considered in a design are:

(1) Direct actions experienced by the structure due to self weight and external actions.
(2) Imposed deformations such as that due to temperature and settlement.
IS 800-2007, classified various actions in the following three groups:
   (a) *Permanent Actions (Q_p):* Actions due to self weight and fixed equipment etc.
   (b) *Variable Actions (Q_v):* Actions during construction and service stage such as imposed loads, wind loads and earthquake loads etc.
   (c) *Accidental Actions (Q_a):* Actions expected due to explosions and impact of vehicles etc.

Characteristic Actions (Q_c)

The characteristic actions (Q_c) are defined as the values of different actions which are not expected to be exceeded with more than 5 percent probability, during the life of the structure. One can work out these actions by statistical analysis, in all special cases, subject to minimum values specified in codes.

In the absence of statistical analysis, the loads presented in IS 875 and other special codes may be considered characteristic loads.

Design Actions (Loads)

Noting the importance of safety in civil engineering structures and the uncertainties involved in the analysis, design and construction, code specifies taking design actions as partial safety factor times the characteristic actions. The partial safety factors specified by code for limit state of strength and serviceability differ. The partial safety factors for loads are as given in Table 2.1 and design load Q_d is to be found as

$$Q_d = \sum_k \gamma_{fk}\, Q_{ck}$$

where γ_{fk} is partial safety factor for kth load.

Table 2.1 Partial safety factors for loads, γ_f for limit state
[Table 4 of IS 800-2007]

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL		WL/EL	AL	DL	LL		WL/EL
		Leading	Accom-panying				Leading	Accom-panying	
DL + LL + CL	1.5	1.5	1.05	–	–	1.0	1.0	1.0	–
DL + LL + CL	1.2	1.2	1.05	0.6	–	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2	–	–	–	–	–
DL + WL/EL	1.5 (0.9)	–	–	1.5	–	1.0	–	–	1.0
DL + ER	1.2 (0.9)	1.2	–	–	–	–	–	–	–
DL + LL + AL	1.0	0.35	0.35	–	1.0	–	–	–	–

Notes:
1. Lower value of γ_f for DL is to be considered if DL causes higher value for load effect and lower value is to be considered, if DL contributes to the stability of structure against overturning while designing for stability.
2. DL = Dead Load, LL = Imposed Load, WL = Wind Load, CL = Crane Load, AL = Accidental Load, ER = Erection Load and EL = Earthquake Load.

2.4 DESIGN STRENGTH

In using the strength value of a material for design, the following uncertainties should be accounted:

(a) Possibility of unfavourable deviation of material strength from the characteristic value.
(b) Possibility of unfavourable variation of member sizes.
(c) Possibility of unfavourable reduction in member strength due to fabrication and tolerances, and
(d) Uncertainty in the calculation of strength of materials.

Hence IS 800-2007, recommends reduction in the strength of materials by a partial safety factor γ_m which is defined as

$$\gamma_m = \frac{S_u}{S_d} \quad \text{i.e.} \quad S_d = \frac{S_u}{\gamma_m}$$

where S_u – ultimate strength
and S_d – design strength

These values are as shown in Table 2.2.

2.5 DEFLECTION LIMITS

Deflection limits are specified from the consideration that excess deformations do not cause damage to finishing. Deflections are to be checked to adverse but realistic combination of service loads and their arrangement. Elastic analysis may be used to find deflection. Design load for this purpose is the same as characteristic load (i.e. partial safety factor γ_f = 1.0) except when apart from DL, LL, CL and some more imposed loads are considered (Refer Table 2.1).

The deflection limits specified by IS 800:2007 are as shown in Table 2.3 [Refer next page].

Table 2.2 Partial safety factors for materials γ_m
[Table 5 of IS 800-2007]

Sl. No.	Definitions	Partial Safety Factor	
1	Resistance, governed by yielding (γ_{mo})	1.10	
2	Resistance of member to buckling (γ_{mo})	1.10	
3	Resistance governed by ultimate stress (γ_{ml})	1.25	
4	Resistance of connections	Shop fabrication	Field fabrication
	a) Bolts-friction type γ_{mf}	1.25	1.25
	b) Bolts-bearing type γ_{mb}	1.25	1.25
	c) Rivets-γ_{mr}	1.25	1.25
	d) Welds-γ_{mw}	1.25	1.50

2.6 OTHER SERVICEABILITY LIMITS

Apart from deflection requirement, the design should also satisfy the following serviceability limits:

(a) Vibration limit
(b) Durability consideration
(c) Fire resistance.

Vibration Limit

Though most of the structures are designed for strength and then checked for deflection limits, some of the structures need check for vibration limits. The structures the floors of which support machineries, the flexible structures (with height to effective width ratio exceeding 5:1) etc., should be investigated for vibration under dynamic loads. In such cases there are possibilities of resonance, fatigue failures. IS 800-2007 gives a set of guidelines to take care of vibration limits in its Annex C.

Durability Considerations

The following factors affect the durability of a steel structure:

(a) Environment
(b) Degree of exposure
(c) Shape of the member and the structural detail
(d) Protective measures
(e) Ease of maintenance

Table 2.3 Deflection limits
[Table 6 in IS 800]

Type of Building (1)	Deflection (2)	Design Load (3)	Member (4)	Supporting (5)	Maximum Deflection (6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50t)	Gantry	Crane	Span/1000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane + wind	Column/ frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Buildings	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter-storey drift	–	Storey height/300

A designer should refer to the IS code provisions given in section 15 of IS 800-2007 and also to specialised literature on durability.

Fire Resistance

A steel structure should have sufficient fire resistance level (FRL) specified in terms of minutes depending upon the purpose for which the structure is used and the time taken to evacuate in case of fire. For detailed specifications a designer may refer section 16 of IS 800-2007 along with IS 1641, IS 1642, IS 1643 and any other specialised literature on fire resistance.

2.7 STABILITY CHECKS

After designing a structure for strength and stability, it should be checked for instability due to overturning, uplift or sliding under factored loads. In checking for instability disturbing forces should be taken as design loads and stabilising forces may be taken as design loads (factored loads) with lesser factor of safety (0.9) as specified in Table 2.1.

A structure should be adequately stiff against sway and fatigue also.

In the chapters to follow now onwards, design principles are made clear from the point of limit states of strength and deflections. In most of the buildings these are the predominant limit states, but in all important and special buildings, a designer has to ensure that other limit states are not exceeded.

* * *

Questions

1. Explain the principles of
   (a) Working stress method of design
   (b) Ultimate load design and
   (c) Limit state design.
2. Explain how limit state method differs from working stress method of design.
3. Explain how limit state design differs from ultimate load design.
4. Explain the following terms
   (a) Partial safety factor for loads
   (b) Partial safety factor for material strength.
5. Distinguish between
   (a) Factor of safety and partial safety factor
   (b) Characteristic loads and design (factored) loads.

3

BOLTED CONNECTIONS

As steel structures are to be formed by connecting available standard sections there is need for designing the following connections:

(a) different sections to form the required composite section of a member (e.g. connecting plates, angles, channels, I-sections etc.)
(b) different members at their ends (e.g. secondary beams to main beams, beams to columns, columns to footing or members of trusser etc.).

The design of connections is very important because the failure of joint is sudden and catastrophic.

The following three types of connections may be made in steel structures:

(a) Riveted
(b) Bolted
(c) Welded.

In this chapter brief introduction is given to riveted connection and detailed design procedure is explained for bolted connections. The design of welded connections is explained in chapter 4.

3.1 RIVETED CONNECTION

Riveting is a method of joining together pieces of metal by inserting ductile metal pins called rivets into holes of pieces to be connected and forming a head at the end of the rivet to prevent each metal piece from coming out. Figure 3.1 shows connecting two plates by riveting.

Rivet holes are made in the structural members to be connected by punching or by drilling. The size of rivet hole is kept slightly more (1.5 to 2 mm) than the size of rivet. After the rivet holes in the members are matched, a red hot rivet is inserted which has a shop made head on one side and the length of which is slightly more than the combined thicknesses of the members to be connected. Then holding red hot rivet at shop head end, hammering is made. It results into expansion of the rivet to completely fill up the rivet hole and also into formation at driven head. Desired shapes can be given to the driven head. The riveting may be in the workshops or in the field.

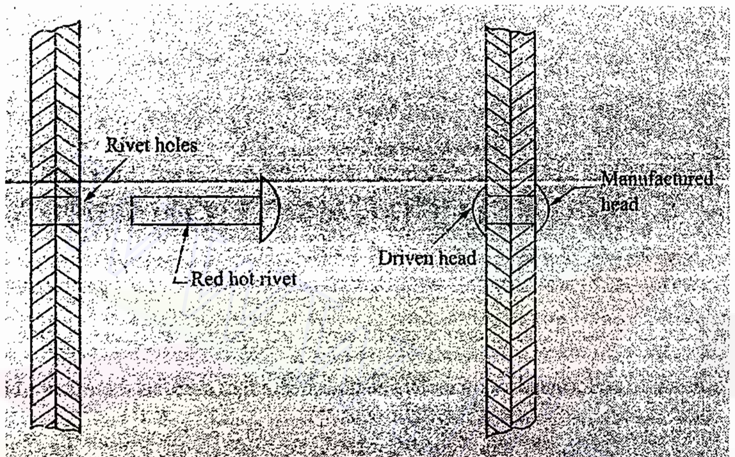

Figure 3.1 Riveting.

Riveting has the following disadvantages:

(a) It is associated with high level of noise pollution.
(b) It needs heating the rivet to red hot.
(c) Inspection of connection is a skilled work.
(d) Removing poorly installed rivets is costly.
(e) Labour cost is high.

Production of weldable quality steel and introduction of high strength friction grip bolts (HSFG) have replaced use of rivets. Design procedure for riveted connections is same as that for bolted connection except that the effective diameter of rivets may be taken as rivet hole diameter instead of nominal diameter of rivet. Hence riveted connection is not discussed further in this chapter.

3.2 BOLTED CONNECTIONS

A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut. Bolts are used for joining together pieces of metals by inserting them through holes in the metal and tightening the nut at the threaded ends. Figure 3.2 shows a typical bolt.

Bolts are classified as:

(a) Unfinished (Black) Bolts
(b) Finished (Turned) Bolts
(c) High Strength Friction Grip (HSFG) Bolts.

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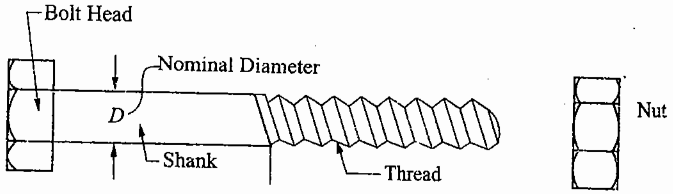

Figure 3.2 Bolt and nut.

Unfinished/Black Bolts

These bolts are made from mild steel rods with square or hexagonal head. The shank is left unfinished i.e. rough as rolled. Though the black bolts of nominal diameter (diameter of shank) of sizes 12, 16, 20, 22, 24, 27, 30 and 36 mm are available, commonly used bolt diameters are 16, 20, 24, 30 and 36 mm. These bolts are designated as M16, M20, M24, etc. IS 1364 (part 1) gives specifications for such bolts. In structural elements to be connected holes are made larger than nominal diameter of bolts. As shanks of black bolts are unfinished, the bolt may not establish contact with structural member at entire zone of contact surface. Joints remain quite loose resulting into large deflections. The yield strength of commonly used black bolts is 240 N/mm^2 and ultimate strength 400 N/mm^2. These bolts are used for light structures under static loads such as trusses, bracings and also for temporary connections required during erections.

Finished/Turned Bolts

These bolts are also made from mild steel, but they are formed from hexagonal rods, which are finished by turning to a circular shape. Actual dimension of these bolts are kept 1.2 mm to 1.3 mm larger than the nominal diameter. As usual the bolt hole is kept 1.5 mm larger than the nominal diameter. Hence tolerance available for fitting is quite small. It needs special methods to align bolt holes before bolting. As connection is more tight, it results into much better bearing contact between the bolts and holes. These bolts are used in special jobs like connecting machine parts subjected to dynamic loadings. IS 3640 covers specifications for such bolts.

High Strength Friction Grip (HSFG) Bolts

The HSFG bolts are made from high strength steel rods. The surface of the shank is kept unfinished as in the case of black bolts. These bolts are tightened to a proof load using calibrated wrenches. Hence they grip the members tightly. In addition nuts are provided by using clamping devices. If the joint is subjected to shearing load it is primarily resisted by frictional force between the members and washers. The shank of the bolt is not subjected to any shearing. This results into no-slippage in the joint. Hence such bolts can be used to connect members subjected to dynamic loads also. The successful introduction of HSFG bolt resulted into replacement of rivets. IS 3747 specifies various dimensions for such bolts and for their washers and nuts. Commonly available nominal diameter of HSFG bolts are 16, 20, 24, 30 and 36 mm.

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3.3 CLASSIFICATION OF BOLTS BASED ON TYPE OF LOAD TRANSFER

On the basis of load transfer in the connection bolts may be classified as

(a) Bearing Type
(b) Friction Grip Type.

Unfinished (black) bolts and finished (turned) bolts belong to bearing type since they transfer shear force from one member to other member by bearing, whereas HSFG bolts belong to friction grip type since they transfer shear by friction.

Advantages of HSFG Bolts Over Bearing Type Bolts

HSFG bolts have the following advantages over unfinished or finished bolts:

1. Joints are rigid i.e., no slip takes place in the joint.
2. As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses.
3. High static strength due to high frictional resistance.
4. High fatigue strength since nuts are prevented from loosening and stress concentrations avoided due to friction grip.
5. Smaller number of bolts result into smaller sizes of gusset plates.

Disadvantages of HSFG Bolts

The following are the disadvantages of HSFG bolts over bearing type bolts:

1. Material cost is high.
2. The special attention is to be given to workmanship especially to give them right amount of tension.

3.4 ADVANTAGES AND DISADVANTAGES OF BOLTED CONNECTIONS

The following are the **advantages** of bolted connections over riveted or welded connections:

1. Making joints is noiseless.
2. Do not need skilled labour.
3. Needs less labour.
4. Connections can be made quickly.
5. Structure can be put to use immediately.
6. Accommodates minor discrepancies in dimensions.
7. Alterations, if any, can be done easily.
8. Working area required in the field is less.

The **disadvantages** of unfinished (black) bolt connections are listed here. However it may be noted that most of these disadvantages are overcome by using HSFG bolts.

1. Tensile strength is reduced considerably due to stress concentrations and reduction of area at the root of the threads.
2. Rigidity of joints is reduced due to loose fit, resulting into excessive deflections.
3. Due to vibrations nuts are likely to loosen, endangering the safety of the structures.

3.5 TERMINOLOGY

The following terms used in the bolted connections are defined below:

1. *Pitch of the bolts (p):* It is the centre to centre spacing of the bolts in a row, measured along the direction of load. It is shown as '*p*' in Fig. 3.3.
2. *Gauge Distance (g):* It is the distance between the two consecutive bolts of adjacent rows and is measured at right angles to the direction of load. (Ref. Fig. 3.3)

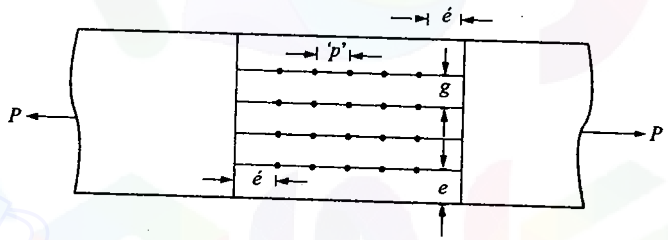


Figure 3.3 Pitch, gauge distance and edge distance.

3. *Edge Distance (e):* It is the distance of centre of bolt hole from the adjacent edge of plate (Ref. Fig. 3.3).
4. *End Distance (e'):* It is the distance of the nearest bolt hole from the end of the plate (Ref. Fig. 3.3).
5. *Staggered Distance:* It is the centre to centre distance of staggered bolts measured obliquely on the member as shown in Fig. 3.4.

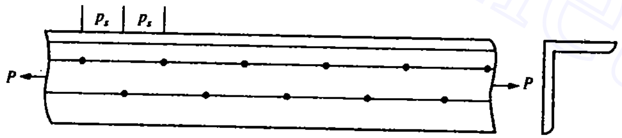


Figure 3.4 Bolt distance in staggered bolts.

3.6 IS 800-2007 SPECIFICATIONS FOR SPACING AND EDGE DISTANCES OF BOLT HOLES

1. Pitch 'p' shall not be less than $2.5d$, where 'd' is the nominal diameter of bolt.
2. Pitch 'p' shall not be more than
   (a) $16t$ or 200 mm, whichever is less, in case of tension members [Fig. 3.5],

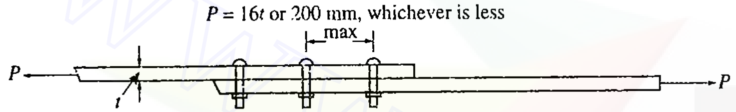


Figure 3.5

   (b) $12t$ or 200 mm, whichever is less, in case of compression members where t is the thickness of thinnest member (Fig. 3.6).

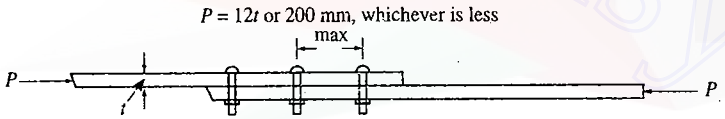
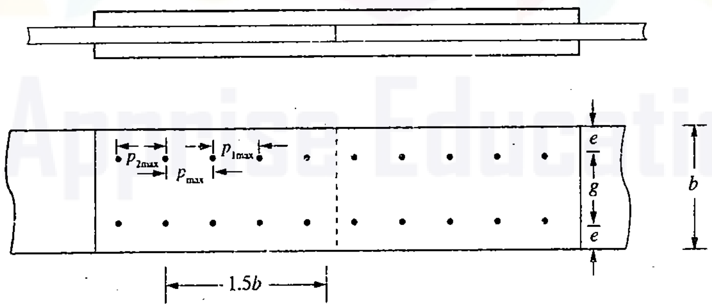


Figure 3.6

   (c) In case of staggered pitch, pitch may be increased by 50 percent of values specified above provided gauge distance is less than 75 mm.
3. In case of butt joints maximum pitch is to be restricted to $4.5d$ for a distance of 1.5 times the width of plate from the butting surface (Ref. Fig. 3.7).



p_{1max} as per clause 3c and $p_{2\,max}$ as per clause 2a or 2b.

Figure 3.7 p_{max} for butt joints.

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4. The gauge length 'g' should not be more than $100 + 4t$ or 200 mm whichever is less.
5. Minimum edge distance shall not be
   (a) Less than 1.7 × hole diameter in case of sheared or hand flame cut edges
   (b) Less than 1.5 × hole diameter in case of rolled, machine flame cut, sawn and planed edges.
6. Maximum edge distance (e) should not exceed

   (a) $12t\epsilon$, where $\epsilon = \sqrt{\dfrac{250}{f_y}}$ and t is the thickness of thinner outer plate

   (b) $40 + 4t$, where t is the thickness of thinner connected plate, if exposed to corrosive influences
7. Apart from the required bolt from the consideration of design forces, additional bolts called tacking fasteners should be provided as specified below.
   (a) If value of gauge length exceeds after providing design fasteners at maximum edge distances tacking rivets should be provided
      (i) At 32 t or 300 mm, whichever is less, if plates are not exposed to weather
      (ii) At 16 t or 200 mm, whichever is less, if plates are exposed to weather.
8. In case of a member made up of two flats, or angles or tees or channels, tacking rivets are to be provided along the length to connect its components as specified below:
   (a) Not exceeding 1000 mm, if it is tension member
   (b) Not exceeding 600 mm, if it is compression member

This situation is shown in Fig. 3.8

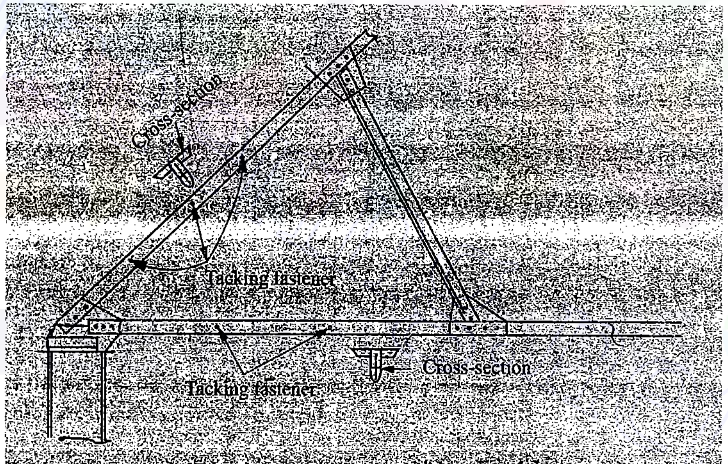

Figure 3.8 Tacking fasteners required in a truss.

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3.7 TYPES OF BOLTED CONNECTIONS

Types of joints may be grouped into the following two:

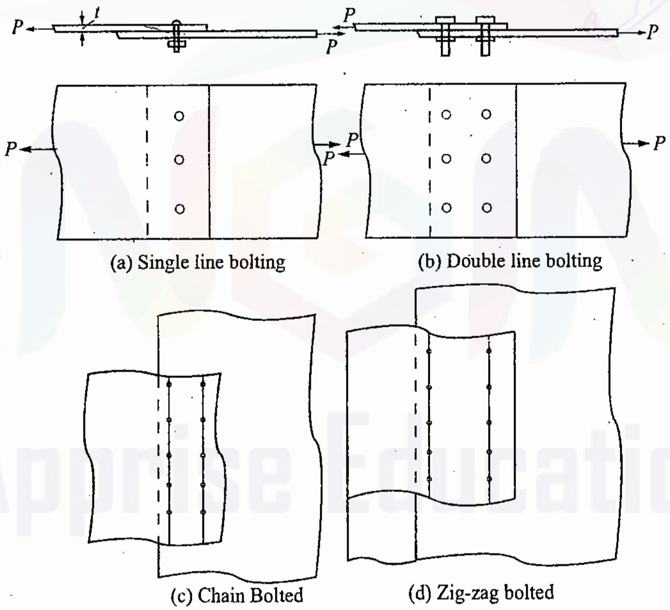
(a) Lap joint
(b) Butt joint

(a) Lap Joint

It is the simplest type of joints. In this the plates to be connected overlap one another. Figure 3.9 shows a typical lap joints.

(b) Butt Joint

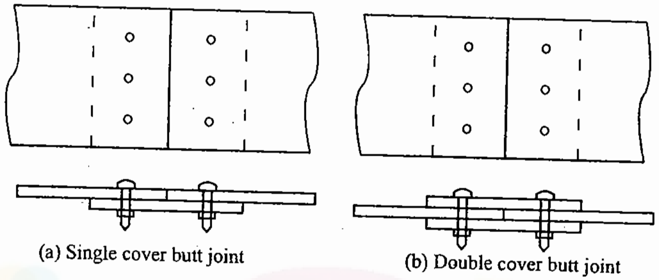
In this type of connections, the two main plates abut against each other and the connection is made by providing a single cover plate connected to main plate or by double cover plates, one on either side connected to the main plates (Ref. Fig. 3.10).



(a) Single line bolting (b) Double line bolting

(c) Chain Bolted (d) Zig-zag bolted

Figure 3.9 Types of lap joints.



(a) Single cover butt joint (b) Double cover butt joint

Figure 3.10 Butt joints.

3.8 TYPES OF ACTIONS ON FASTENERS

Depending upon the types of connections and loads, bolts are subjected to the following types of actions:

(a) Only one plane subjected to shear (single shear)
(b) Two planes subjected to shear (double shear)
(c) Pure tension
(d) Pure moment
(e) Shear and moments in the plane of connection
(f) Shear and tension.

These cases are shown in Fig. 3.11.

3.9 ASSUMPTIONS IN DESIGN OF BEARING BOLTS

The following assumptions are made in the design of bearing (finished or unfinished) bolted connections:

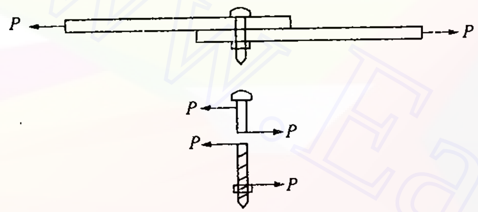
1. The friction between the plates is negligible
2. The shear is uniform over the cross-section of the bolt
3. The distribution of stress on the plates between the bolt holes is uniform
4. Bolts in a group subjected to direct loads share the load equally
5. Bending stresses developed in the bolts is neglected.

Assumption 1 is not correct because friction exists between the plates as they are held tightly by bolts. But this assumption results on safer side in the design.

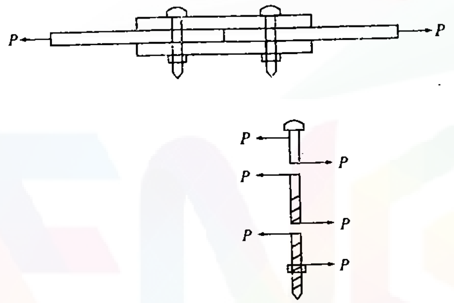
Actual stress distribution in the plate is not uniform in working conditions. Stresses are very high near bolt holes. But with increase in load the fibres near the hole start yielding and hence stresses at other parts

start increasing. At failure, the stress distribution is uniform and the ultimate load carrying capacity is given by the net area times the yield stress.

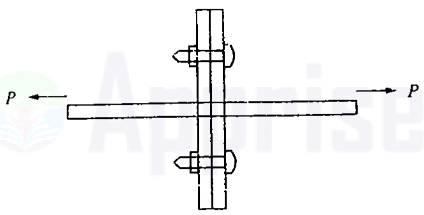
The fourth assumption is questionable. The bolts far away from centre of gravity of bolt groups are subject to more loads. In the ultimate stage all rivets have to fail, till then redistribution of load will be taking place. Hence the assumption is not completely wrong. IS 800-2007 permits this assumption for short joints (distance between first and the last bolt in the direction of load being less than ($5 \times d$)). For long a reduction factor has been recommended for finding the strength of joint.



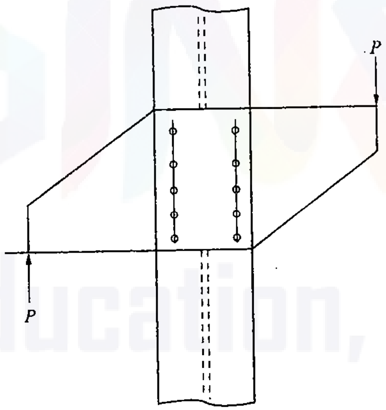
(a) Shear plane on thread



(b) Two planes subject to shear

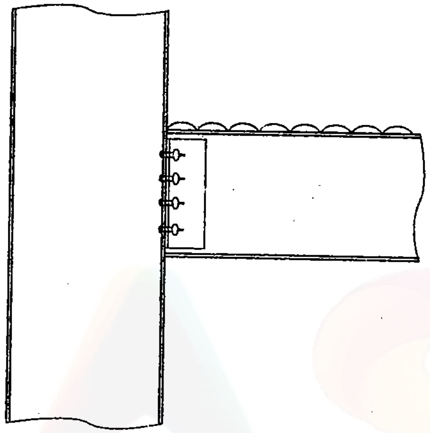


(c) Bolts in direct tension

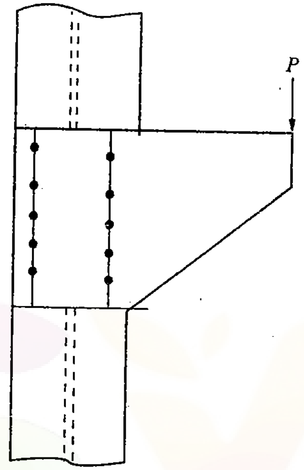


(d) Bolts resisting pure moment

Figure 3.11 Actions on fasteners.



(e) Bolts subject to shear and tension



(f) Joint subject to shear and moment in its plane

Figure 3.11 (continued)

3.10 PRINCIPLES OBSERVED IN THE DESIGN

The following principles are observed in the design of connections:

1. The centre of gravity of bolts should coincide with the centre of gravity of the connected members.
2. The length of connection should be kept as small as possible.

3.11 DESIGN TENSILE STRENGTH OF PLATES IN A JOINT

Plates in a joint made with bearing bolts may fail under tensile force due to any one of the following:

1. Bursting or Shearing of the edge (Fig. 3.12).
2. Crushing of Plates (Fig. 3.13).
3. Rupture of Plates (Figs. 3.14 and 3.15).

The bursting or shearing and crushing failures are avoided if the minimum edge/end distances as per IS 800-2007 recommendations are provided.

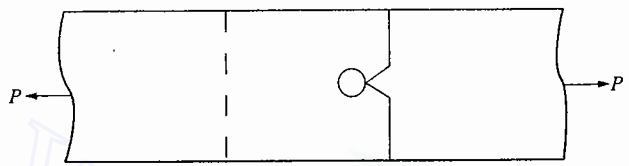


Figure 3.12 Bursting or shearing of plates.

Figure 3.13 Crushing of plates.

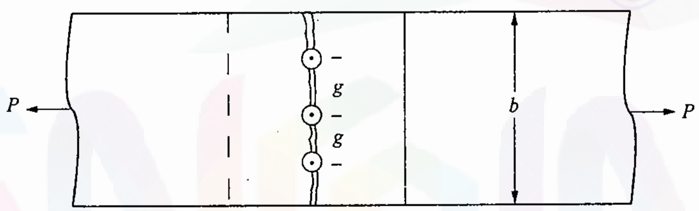


Figure 3.14 Rupture of plate.

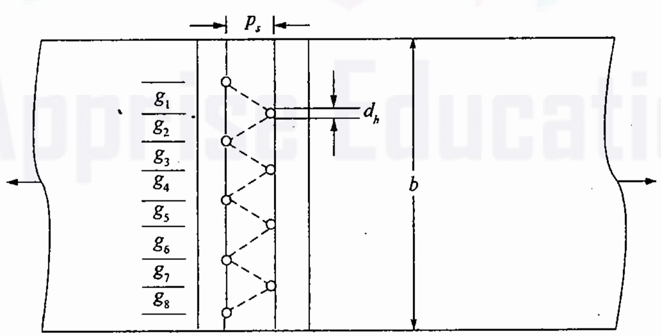


Figure 3.15 Joint with staggered pitch.

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If the minimum distances are ensured in a joint, the design tensile strength of plate in the joint is the strength of the thinnest member against rupture. This strength is given by

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$$

where

γ_{ml} = partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of the material

A_n = net effective area of the plate at critical section, which is given by

$$A_n = \left[b - n d_0 + \sum \frac{p_{si}^2}{4 g_i} \right] t$$

where

b = width of plate

t = thickness of thinner plate in joint

d_0 = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case of directly punched holes)

g = gauge lengths between the bolt holes (Ref. Fig. 3.14)

p_s = staggered pitch length between lines of bolt holes

n = number of bolt holes in the critical section

i = subscript for summation of all inclined legs

It may be noted that, if there is no staggering, $p_{si} = 0$ and hence, $A_n = (b - nd_0)t$, which is the critical section shown in Fig. 3.14.

3.12 DESIGN STRENGTH OF BEARING BOLTS

The design strength of bearing bolts under shear is the least of the following:

(a) Shear capacity (strength)
(b) Bearing capacity (strength)

(a) Shear Capacity (Strength) of Bearing Bolts in a Joint

Design strength of the bolt, V_{dsb} is given by

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

where V_{nsb}, nominal shear capacity of bolt and γ_{mb} = partial safety factor of material of bolt.

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In the above expression V_{nsb} is given by

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

where,

f_{ub} = ultimate tensile strength of the bolt

n_n = number of shear planes with threads intercepting the shear plane

n_s = number of shear planes without threads intercepting the shear plane

A_{sb} = nominal shank area of the bolt, and

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter of the thread.

$A_{nb} = \frac{\pi}{4}(d - 0.9382p)^2$, where p is pitch of thread

$\approx 0.78\frac{\pi}{4}d^2$ for ISO threads.

Note: In Fig. 3.11(a), per bolt, $n_n = 1$ and $n_s = 0$ and in Fig. 3.11(b), per bolt, $n_n = 1$ and $n_s = 1$.

Reduction Factors for Shear Capacity of Bolts

The code suggests the use of reduction factors for shear capacity in the following situations:

(i) If the joint is too long
(ii) If the grip length is large
(iii) If the packing plates of thickness more than 6 mm are used.

(i) *Reduction Factor for Long Joints* (β_{lj})

If the distance between the first and the last bolt in the joint (l_j) measured in the direction of load exceeds 15d, the shear capacity V_{db} shall be reduced by the factor β_{lj} given by

$$\beta_{lj} = 1.075 - 0.005\frac{l_j}{d}$$

subjected to the limits $0.75 \le \beta_{lj} \le 1.0$, where d is nominal diameter of bolt.

(ii) *Reduction Factor if Grip Length is Large* (β_{lg})

If the total thickness of the connected plates exceed 5 times the diameter d of bolts, the design shear capacity V_{db}, shall be reduced by

$$\beta_{lg} = \frac{8d}{3d + l_g}$$

subject to conditions maximum value = β_{lj}, where l_g = grip length = total thickness of the connected plates. In no case l_g be greater than 8d.

(iii) *Reduction Factor if Packing Plates are Used* (β_{pk})

If packing plates of thickness more than 6 mm are used in the joint, then shear capacity is to be reduced by a factor

$$\beta_{pk} = 1 - 0.0125\, t_{pk}$$

where t_{pk} = thickness of the thicker packing in mm.

Thus bearing capacity of the bolts in shear is $\frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\beta_{lj}\,\beta_{lg}\,\beta_{pk}$

(b) Bearing Capacity of Bolts (V_{dpb})

IS 800-2007 suggests the following procedure to find bearing strength of bolts:

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

where V_{dpb} = design bearing strength
V_{npb} = nominal bearing strength
and γ_{mb} = partial safety factor of material = 1.25 (Table 2.2).

Nominal shearing strength may be found from the following relation:

$$V_{npb} = 2.5\, K_b\, d t f_u$$

where K_b is smaller of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

in which e, p = end and pitch distances.

d_0 = diameter of hole.

f_{ub}, f_u = ultimate tensile stress of the bolt and plate.

d = nominal diameter of the bolt.

t = summation of the thickness of the connected plates experiencing bearing stress in the same direction. If bolts are counter sunk, it is to be reduced by the half depth of counter sinking.

3.13 DESIGN PROCEDURE WITH BEARING TYPE BOLTS SUBJECT TO SHEARING FORCES

Determine the design (factored) action acting on the joint. Then select connection with suitable diameter of the bolts. Determine the strength of connection and ensure that design strength is not less than the design action. The following information is useful in the design of joint:

(1) Diameter of bolt hole:

Nominal size of bolts (d) in mm	12	14	16	20	22	24	30	36
Diameter of bolt hole (d_0) in mm	13	15	18	22	24	26	33	39
Outer diameter of washers in mm	–	–	30	37	–	44	56	60

(2) Area of bolt at root (A_{nb}):

$$A_{nb} \approx 0.78\, A_{sb}$$

where A_{sb} = area of bolt at shank $= \dfrac{\pi}{4} d^2$

(3) Properties of materials of bolts: Commonly used bolts have the following material properties (IS 1367):

Grade 4.6	$f_{yb} = 240$ MPa	$f_{ub} = 400$ MPa
Grade 4.8	$f_{yb} = 320$ MPa	$f_{ub} = 420$ MPa
Grade 5.6	$f_{yb} = 300$ MPa	$f_{ub} = 500$ MPa
Grade 5.8	$f_{yb} = 400$ MPa	$f_{ub} = 520$ MPa

(4) Properties of rolled steel sections: These values have been shown in Appendix.

3.14 EFFICIENCY OF A JOINT

It is defined as the ratio of strength of joint and strength of solid plate in tension. It is usually expressed in percentage. Thus,

$$\text{efficiency } \eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$$

Strength of solid plate is less in yielding compared to tearing of solid plate. For example, consider Fe 415 plates.

$f_y = 250$ N/mm^2 $\quad f_u = 410$ N/mm^2

$\gamma_{mo} = 1.1 \quad \gamma_{ml} = 1.25$

∴ Design strength of solid plate per unit area

(a) in yielding is $\dfrac{250}{1.1} \times 1 = 227.27$ N/mm^2

(b) in rupture is $\dfrac{0.9 \times 410}{1.25} \times 1 = 295.2$ N/mm^2

Hence strength of solid plate is governed by its strength in yielding.

Strength of joint is the smaller of strength in shear and strength in bearing.

Example 3.1

Find the efficiency of the lap joint shown in Fig. 3.16. Given: M20 bolts of grade 4.6 and Fe 410 (E 250) plates are used.

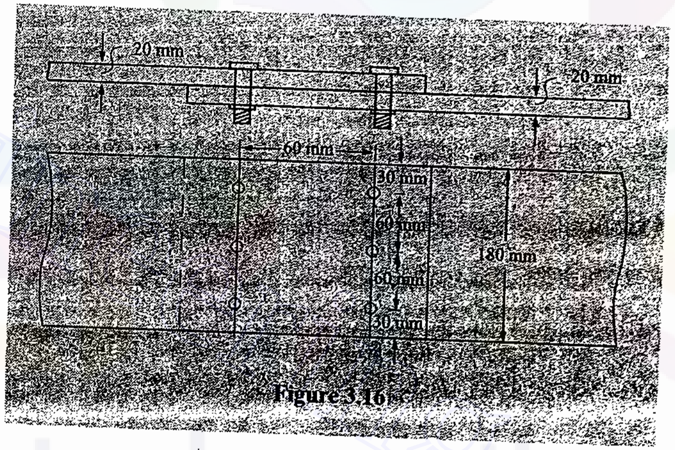

Figure 3.16

Solution:

For M20 bolts of grade 4.6,

diameter of bolt, $d = 20$ mm

diameter of bolt hole, $d_0 = 22$ mm

Ultimate strength $f_{ub} = 400$ MPa

Partial safety factor, $\gamma_{mb} = 1.25$

For Fe 410 (E 250) plates,

Ultimate stress, $f_u = 410$ MPa

Partial safety factor, $\gamma_{ml} = 1.25$

Strength of plates in the joint:

Thickness of thinner plate, $t = 20$ mm

Width of plate $b = 180$ mm

There is no staggering $p_{si} = 0$

Number of bolt holes in the weakest section = 3

$\therefore$ Net area at weakest section

$$A_u = [b - nd_0 + 0]\, t$$
$$= [180 - 3 \times 22] \times 20 = 2280 \text{ mm}^2$$

Design strength of plates in the joint

$$T_{dn} = \frac{0.9 f_u A_n}{\gamma_{ml}} = \frac{0.9 \times 410 \times 2280}{1.25}$$
$$= 673056 \text{ N} = 673.056 \text{ kN.}$$

Strength of Bolts:

Total number of bolts = 6

(i) *Design Strength in Shear:*

Number of shear planes at thread $n_n = 1$ per bolt.

Number of shear planes at shank $n_s = 0$ per bolt.

$\therefore$ Total $n_n = 1 \times 6 = 6$ and total $n_s = 0$.

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245 \text{ mm}^2$$

There are no reduction factors i.e. $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$

$\therefore$ Nominal shear strength,

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$
$$= \frac{400}{\sqrt{3}} (6 \times 245 + 0) = 339482 \text{ N} = 339.482 \text{ kN}$$

$\therefore$ Design strength in shear,

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{339.482}{1.25} = 271.586 \text{ kN}$$

(ii) *Design Strength in Bearing:*

Nominal strength

$$V_{npb} = 2.5\, K_b d t f_u$$

where K_b = least of the following:

(a) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$

(b) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6591$

(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(d) 1.0.

Note: Edge distance provided is less. Hence it is critical in this case.

$K_b = 0.4545$

$\therefore V_{npb} = 2.5 \times 0.4545 \times 20 \times 20 \times 410 = 186345$ N per bolt

Design strength $= \frac{V_{npb}}{\gamma_{mb}} = \frac{186345}{1.25} = 149076$ N

$\therefore$ Design strength of joint $= 6 \times 149076 = 894456.8$ N

$= 894.456$ kN

$\therefore$ Design strength of bolts in joint $= 271.586$ kN $< T_{dn}$.

$\therefore$ Strength of joint $= 271.586$ kN.

Efficiency of Joint:

Area of solid plate $= 180 \times 20 = 3600 \text{ mm}^2$.

$\therefore$ Design strength of solid plate

$$= \frac{f_y}{\gamma_{mb}} \times A_g = \frac{250}{1.1} \times 3600 = 818181.8 \text{ N} = 818.182$$

$\therefore$ Efficiency of the joint $= \dfrac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$ percent

$$= \frac{271.586}{818.182} \times 100 = 33.19\%$$

Example 3.2

Find the efficiency of the joint, if in the above example instead of lap, butt joint is made using two cover plates each of size 12 mm and 6 numbers of bolts on each side.

Solution:

In this case strength of plates and strength of bolts in bearing are same as in example 3.1. Strength in shear is different, since in each bolt a section in root and another section at shank resist shear. Thus in this case total number of section resisting shear at shank $n_s = 6$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2, \quad A_{nb} = 0.78 \ \frac{\pi}{4} \ d^2 = 245 \text{ mm}^2$$

$\therefore$ Nominal shear strength $= \dfrac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$

$$= \frac{400}{\sqrt{3}} (6 \times 245 + 6 \times 314.16)$$

$= 774795$ N

$= 774.795$ kN

$\therefore$ Design shear strength $= \dfrac{774.795}{1.25} = 619.836$ kN

Design strength in bearing $= 894.456$ kN (see example 3.1)

$\therefore$ Design strength of bolt $= 619.836$ kN

Design strength of plates T_{dn} $= 673.096$ kN

Note: Since total thickness of cover plates = 2 × 12 = 24 mm which is more than thickness of the plates, strength of plates is the strength of main plates only.

$\therefore$ Design strength of joint = 619.836 kN

Design strength of solid pla $= \dfrac{250}{1.1} \times 180 \times 20 = 818182 \text{ N} = 818.182 \text{ kN}$

$\therefore$ Efficiency of joint $= \dfrac{619.836}{818.182} \times 100$

$= 75.76\%$ **Answer**

Example 3.3

A boiler shell is made up of 14 mm thick Fe:415 plates. If the joint is double bolted lap joint with M16 bolts of grade 4.6 at distances of 50 mm, determine the design strength of the joint per pitch width. Is it a safe design if the internal diameter of bolt is 1 m and steam pressure is 1.2 MPa?

Solution:

Since the boiler is subjected to hoop tension, the pitch of joint is as shown is Fig. 3.17. In such cases strength is worked out per gauge width of joint and checked for safety and efficiency.

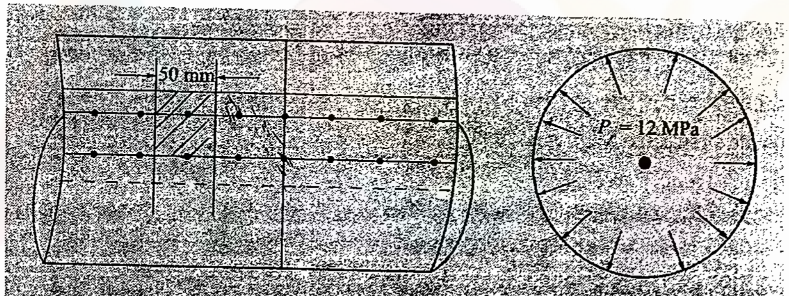

Figure 3.17

Strength of plate per 50 mm width:

Diameter of bolts $d = 16$ mm.

$\therefore$ Diameter of bolt hole $d_0 = 18$ mm.

Strength of plate per 50 mm width:

$t = 14$ mm, $\quad b = p = 50$ mm, $\quad f_u = 410$ MPa

No. of bolts in double bolted joint per 50 mm width $n = 1$

$\therefore A_n = (50 - 1 \times 18) \times 14 = 448 \text{ mm}^2$

$\therefore$ Design strength of plate per 50 mm width.

$$T_{dn} = \frac{0.9 \times 410 \times 448}{1.25} = 132250 \text{ N} = 132.250 \text{ kN}$$

Strength of bolts per 50 mm width:

Since it is lap joint, shear planes at shanks = 0. As there are two bolts per pitch width considered, $n_n = 2$

Area of bolt at roots $= 0.78 \times \frac{\pi}{4}(16)^2$

$= 156.83 \text{ mm}^2$

$\therefore$ Ultimate strength $V_{nsb} = \frac{400}{\sqrt{3}}(0 + 2 \times 156.83)$

$= 72436$ N

$\therefore$ Design strength, $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{72436}{1.25} = 57949 \text{ N} = 57.949 \text{ kN}$

Design strength in bearing:

Ultimate strength in bearing per bolt:

K_b is least of the following:

(a) $\frac{e}{3d_0}$, since 'e' is not given, assume that sufficient edge distance is provided and hence it will not decide K_b.

(b) $\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.6759$

(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(d) 1.0.

$\therefore$ $K_b = 0.6759$.

Ultimate bearing strength of each bolt

$= 2.5\, K_b\, d t f_u$

$= 2.5 \times 0.6759 \times 16 \times 14 \times 410 = 155187 \text{ N} = 155.187 \text{ kN}$

As there are two bolts, design strength of bolts in bearing $= 2 \times 155.187 = 310.374 \text{ kN} > V_{dsb}$

$\therefore$ Design strength of bolts = 57.949 kN.

Strength of joint per 50 mm width is lesser of design strength of bolts (57.949 kN) and strength of plate (132.5 kN).

$\therefore$ Design strength of joint = 57.949 kN per 50 mm width.

To check the safety of joint:

Action of applied force is a hoop strers $= \frac{P_r D}{2t}$

where P_r is applied pressure and D is diameter of boiler.

$= \frac{1.2 \times 1000}{2 \times 14} = 42.857 \text{ N/mm}^2$

$\therefore$ Force per 50 mm length $= 42.857 \times 14 \times 50$

$= 30{,}000$ N

$= 30$ kN

$\therefore$ Factored design action $= 1.5 \times 30 = 45$ kN

Design strength = 57.949 kN > Design action.

Hence the design is safe. **Answer**

Example 3.4

Check the safety of the joint, if in the example 3.3, zig-zag bolting is used as shown in Fig. 3.18.

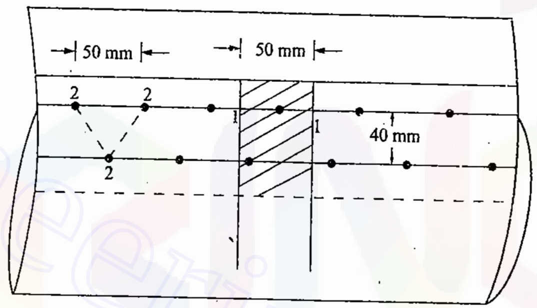

Figure 3.18

Solution:

Consider the strength of joint per 50 mm width as shown in the figure.

$d = 16$ mm $\quad \therefore d_0 = 18$ mm $\quad t = 14$ mm

Number of bolts per 50 mm width = 2

$\therefore$ Design strength of plate:

It should be checked along section (1) – (1) and (2) – (2) as shown in Fig. 3.18.

Net cross sectional area resisting tearing along

(a) Section (1) – (1) $\quad Ax_1 = (b - nd_0)t = (50 - 1 \times 18) \times 14 = 448 \text{ mm}^2$

(b) Section (2) – (2), $An_2 = \left[b - nd_0 + \sum \frac{p_{si}^2}{4g_i}\right]t$

$$= \left[50 - 2\times18 + 2\times\frac{40^2}{4\times25}\right]\times14$$

$$= 644\ \text{mm}^2$$

∴ Section (1) – (1) is weaker.

Hence plate strength

$$T_{dn} = 0.9\times448\times\frac{410}{1.25} = 132250\ \text{N}$$

$$= 132.250\ \text{kN}$$

Strength of bolt per 50 mm width of joint:

a) In shear = 57.949 kN

b) In bearing = 155.187 kN (As in previous example)

∴ Strength ot joint = 57.949 kN

∴ Design action = 45 kN

Design strength (57.949 kN) > Design action (45 kN)

∴ Design is safe.

Example 3.5

Find the maximum force which can be transferred through the double covered butt joint shown in Fig. 3.19. Find the efficiency of the joint also. Given M20 bolts of grade 4.6 and Fe 410 steel plates are used.

Solution:

For M20 bolts of Grade 4.16,

$d = 20$ mm $\quad d_0 = 22$ mm $\quad f_{ub} = 400\ \text{N/mm}^2$.

For grade Fe 410 plates, $f_u = 410\ \text{N/mm}^2$.

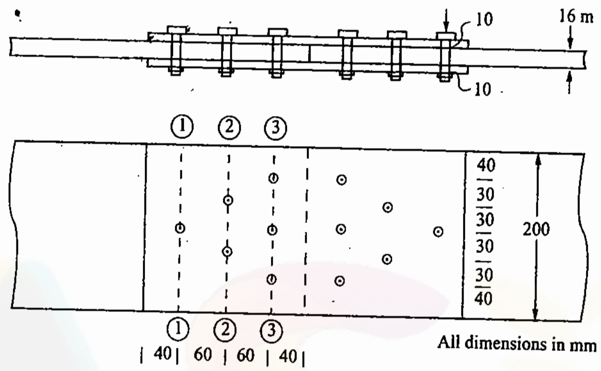

Figure 3.19

∴ Nominal strength of one bolt in shear (double shear)

$$= \frac{f_{ub}}{\sqrt{3}}\left(1\times\frac{\pi}{4}d^2 + 0.78\times\frac{\pi}{4}d^2\right)$$

$$= \frac{400}{\sqrt{3}}(1.78)\times\frac{\pi}{4}20^2$$

$$= 129143\ \text{N}$$

∴ Design strength of one bolt in double shear

$$= \frac{129143}{1.25} = 103314\ \text{N}$$

Design strength of joint in double shear

$$= 6\times103314 = 619886\ \text{N} = 619.886\ \text{kN}$$

Strength of bolts in bearing:

K_b is the least of the following:

$$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$$

∴ For bolts at section (3) – (3), it is least of

$$\frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1.0$$

i.e., $K_b = 0.6061$

$\therefore$ For bolts on section (2)–(2) and (1)–(1), 'e' is large. Hence

$$K_{b1} = K_{b2} = 0.6591, \text{ which is governed by } \frac{p}{3d_0} - 0.25$$

$\therefore$ Nominal strength of six bolts in bearing

$$= 3\,(2.5 \times 0.6061 \times 20 \times 16 \times 410) + 3\,(2.5 \times 0.6591 \times 20 \times 16 \times 410)$$

$$= 1244957 \text{ N}$$

$\therefore$ Design strength in bearing $= \dfrac{1244957}{1.25}$

$$= 995965 \text{ N}$$

$$= 995.965 \text{ kN} > 619.886 \text{ kN}$$

$\therefore$ Strength of bolts in the joint = 619.886 kN and strength of each bolt = 103314 N

Strength of plate:

It is to be checked along all the three sections.

Now, t = 16 mm (least of the thicknesses of cover plates and main plate)

$f_u = 410$ N/mm^2

(a) At section (1) – (1)

$$T_{dn_1} = \frac{0.9\, f_u\, A_n}{1.25} = \frac{0.9 \times 410\,(200 - 22) \times 16}{1.25}$$

$$= 840730 \text{ N}$$

(b) At section (2) – (2)

When this section fails, bolt in section (1) – (1) also has to fail. Hence strength of plate at section (2) – (2)

$$T_{dn_2} = \frac{0.9 \times 410\,(200 - 2 \times 22) \times 16}{1.25} + 103314$$

$$= 840133 \text{ N}$$

At section (3) – (3)

$$T_{dn_3} = \text{Plate strength} + \text{strength of 3 bolts}$$

$$= \frac{0.9 \times 410\,(200 - 3 \times 22) \times 16}{1.25} + 3 \times 103314$$

$$= 942851 \text{ N}$$

$\therefore$ Strength of plate in the joint = 840133 N

$$= 840.133 \text{ kN}$$

$\therefore$ Strength of joint = 619.886 kN

$\therefore$ Maximum design force that can be transferred safely = 619.886 kN.

$\therefore$ Permissible force at working condition $= \dfrac{619.886}{1.5} = 413.257$ kN **Answer**

Design strength of solid plate $= \dfrac{250 \times 200 \times 16}{1.1} = 727272$ N

$$= 727.272 \text{ kN}$$

$\therefore$ Efficiency of the joint $= \dfrac{619.886}{727.272} \times 100 = 85.23\%$ **Answer**

Example 3.6

Two cover plates, 10 mm and 18 mm thick are connected by a double cover butt joint using 6 mm cover plates as shown in Fig. 3.20. Find the strength of the joint. Given M20 bolts of grade 4.6 and Fe 415 plates are used.

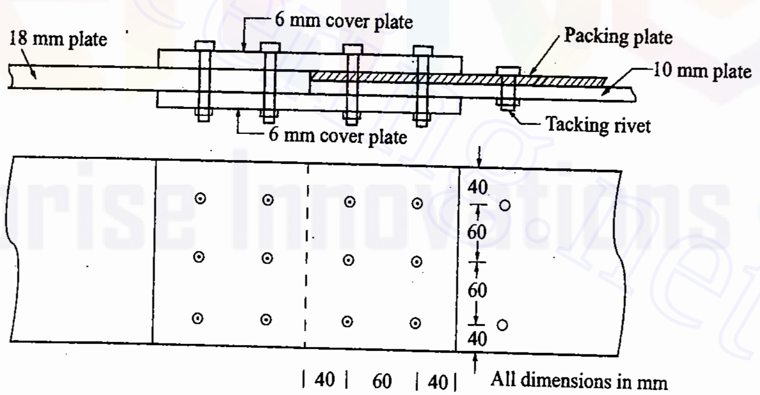

Figure 3.20

Solution:

Note: Strength of tacking rivets are not to be considered in the design.

In this connection packing plate of 8 mm thickness is to be used. Hence there shall be reduction in the shear strength of bolt. The reduction factor is given by

$$\beta_{pk} = (1 - 0.0125\, t_{pk})$$
$$= 1 - 0.0125 \times 8 = 0.9$$

$\therefore$ Nominal shear strength of one bolts in double shear

$$= \beta_{pk} \frac{f_{ub}}{\sqrt{3}}\left(1 \times \frac{\pi}{4} d^2 + 0.78 \times \frac{\pi}{4} d^2\right)$$
$$= 0.9 \times \frac{400}{\sqrt{3}}(1.78) \times \frac{\pi}{4} \times 20^2$$
$$= 116228 \text{ N}$$

Design shear strength of one bolt in shear

$$= \frac{116228}{1.25} = 92982.6 \text{ N}$$

$\therefore$ Design shear strength of 6 bolts in the joint

$$= 6 \times 92982.6 = 557896 \text{ N}$$
$$= 557.896 \text{ kN}$$

Strength of bolts in bearing:

K_b is the minimum of

$$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$$

i.e. $$\frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1.0$$

$$\therefore \quad K_b = 0.6061.$$

$\therefore$ Nominal strength of one bolt in bearing $= 2.5\, K_b\, d t f_u$

$$= 2.5 \times 0.6061 \times 20 \times 10 \times 410$$
$$= 124250.5 \text{ N}$$

Note: Thickness of thinner plate $t = 10$ mm

$\therefore$ Design strength of a bolt $= \frac{124250.5}{1.25} = 99400$ N

Design strength of 6 bolts in bearing $= 6 \times 99400$

$$= 596400 \text{ N.}$$
$$= 596.4 \text{ kN} > 557.896 \text{ kN}$$

$\therefore$ Strength of bolts in connection $= 557.896$ kN.

Strength of plates in the joint = Strength of thinner plate at weakest section.

$\therefore$ Design strength of plate

$$= \frac{0.9 A_n f_u}{\gamma_m} = \frac{0.9 \times [200 - 3 \times 22] \times 10 \times 410}{1.25}$$
$$= 395568 \text{ N}$$
$$= 395.568 \text{ kN} < 597.896 \text{ kN}$$

Design strength of the joint $= 395.568$ kN. **Answer**

Example 3.7

Design a lap joint between the two plates each of width 120 mm, if the thickness of one plate is 16 mm and the other is 12 mm. The joint has to transfer a design load of 160 kN. The plates are of Fe 410 grade. Use bearing type bolts.

Solution:

Using M16 bolts of grade 4.6,

$d = 16$ mm $\quad d_0 = 18$ mm $\quad$ and $\quad f_{ub} = 400$ N/mm^2

Strength of a bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

$\therefore$ Nominal strength of a bolt in shear $= \frac{f_{ub}}{\sqrt{3}}\left(1 \times 0 + 0.78 \times \frac{\pi}{4} d^2\right)$

$$= \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2$$
$$= 36218 \text{ N.}$$

$\therefore$ Design shear strength $= \dfrac{36218}{1.25} = 28974$ N.

Minimum edge distance to be provided $= 1.5 \times 18 = 27$ mm

Minimum pitch to be provided $= 2.5 \times 16 = 40$ mm.

Providing $e = 30$ mm, $p = 40$ mm,

K_b is least of $\dfrac{30}{3\times18}, \dfrac{40}{3\times18} - 0.25, \dfrac{400}{410}$ and 1.0.

i.e., $K_b = 0.4907$

$\therefore$ Nominal bearing strength $= 2.5\, K_b\, d t f_u$

$$= 2.5 \times 0.4907 \times 16 \times 12 \times 400$$

$$= 94222 \text{ N}$$

$\therefore$ Design bearing strength $= \dfrac{94222}{1.25} = 75378$ N

Design strength is lessor of shearing strength and bearig strength.

$\therefore$ Design strength of a M16 bolt $= 28974$ N

$$= 28.974 \text{ kN}$$

Hence to transfer a design force of 160 kN,

No. of bolts required $= \dfrac{160}{28.974} = 5.5$

$\therefore$ Provide 6 bolts. They may be provided in two rows with a pitch of 40 mm as shown in Fig. 3.21.

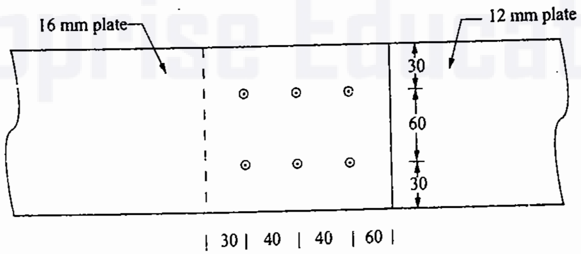

Figure 3.21

Check for the strength of plate:

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_m} = \frac{0.9 \times (120 - 2\times18) \times 12 \times 410}{1.25}$$

$$= 297562 \text{ N} = 297.562 \text{ kN} > 160 \text{ kN safe.}$$

Example 3.8

Design a single bolted double cover butt joint to connect boiler plates of thickness 12 mm for maximum efficiency. Use M16 bolts of grade 4.6. Boiler plates are of Fe 410 grade. Find the efficiency of the joint.

Solution:

$d = 16$ mm $\quad d_0 = 18$ mm $\quad f_{ub} = 400$ N/mm^2

$f_u = 410$ N/mm^2 $\quad t = 12$ mm

Since it is double cover butt joint, the bolts are in double shear one section at shank and another at root.

Nominal strength of a bolt in shear

$$= \frac{400}{\sqrt{3}}\left(1 \times \frac{\pi}{4} \times 16^2 + 1 \times 0.78 \times \frac{\pi}{4} \times 16^2\right)$$

$$= 82651 \text{ N}$$

Design strength in shear $= \dfrac{82651}{1.25} = 66121$ N (a)

Assume bearing strength is more than it. To get maximum efficiency, strength of plate per pitch width should be equated to strength of a bolt.

To avoid failure of cover plates, the total thickness of cover plates should be more than the thickness of main plates. Provide cover plates of 8 mm thicknesses.

Design strength of plate per pitch width

$$= \frac{0.9 \times 410 (p - 18) \times 12}{1.25}$$

$$= 3542.4\,(p - 18) \qquad \text{(b)}$$

Equating (a) to (b) to get maximum efficiency, we get,

$$3542.5\,(p - 18) = 66121$$

$\therefore \quad p = 36.67$ mm.

Minimum pitch $= 2.5 \times 16 = 40$ mm.

$\therefore$ Provide bolts at $p = 40$ mm.

Check for strength of bolt in bearing:

K_b is the minimum of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

Assuming sufficient 'e' will be provided

$$K_b = 0.4907$$

$\therefore$ Design strength of bolt in bearing

$$= \frac{2.5 \times 0.4907 \times 16 \times 12 \times 400}{1.25} = 75372 \text{ N} > 66121 \text{ N}$$

Hence, the assumption that bearing strength is more than design shear is correct.

Since pitch provided is slightly more than required from strength consideration of the plate, the strength of plate is more than the strength of the bolt.

$\therefore$ Design strength of joint per 40 mm width = 66121 N.

Design strength of solid plate per 40 mm width

$$= \frac{250 \times 40 \times 12}{1.1} = 109091 \text{ N.}$$

$\therefore$ Maximum efficiency of joint $= \frac{66121}{109091} \times 100$

$= 60.61\%$ Answer

3.15 ECCENTRIC CONNECTION WITH BEARING BOLTS WHEN LOAD IS IN THE PLANE OF GROUP OF BOLTS

If the line of action of the load does not pass through the centre of gravity of group of bolts, the connection is known as eccentric connection. There are two types of eccentric connections.

(a) Line of action of eccentric load is in the plane of group of bolts [Fig. 3.22].
(b) Line of action of the eccentric load is in the plane perpendicular to the plane of group of bolts [Fig. 3.23].

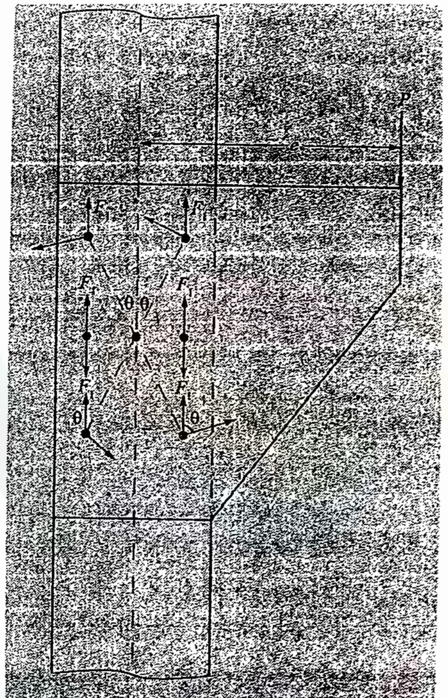

Figure 3.22

The eccentric load P may be replaced by load P acting through centre of gravity of bolts plus moment $P \times e$ acting on the joint. In the connection shown in Fig. 3.22 bolts are subjected to direct shear force and shear force developed to resist moment $P \times e$.

In the eccentric connection shown in Fig. 3.23 bolts are subjected to direct shear and tension in bolts developed to resist bending moment.

Case (a) is discussed in this article whereas case (b) is discussed after presenting codal provisions for tension capacity of bolts and codal specifications for the design of bolts subjected to shear and tension.

Consider the eccentric connection shown in Fig. 3.22 which is subjected to factored (design) load P at an eccentricity 'e'. This load is equivalent to

(a) An axial load P and
(b) a moment $P \times e$.

Let n be the number of bolts in the bracket connection.

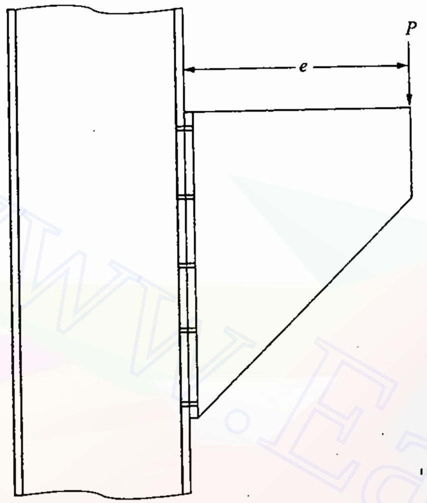

Figure 3.23

$\therefore$ Direct shear force in a bolt $F_1 = \frac{P}{n}$ and it acts in vertical direction.

It is assumed that the bracket is rigid and therefore the force in the bolt due to moment will depend upon its radial distance from the centre of gravity of the bolts and will act at right angles to the radial lines as shown in the figure.

$$F_2 \propto r, \quad \text{or} \quad F_2 = Kr$$

where K is the constant of proportionality.

K can be found by equating resisting moment to applied moment. Thus

$$\sum F_2\, r = P \cdot e$$

$$\sum K\, r^2 = P \cdot e$$

$$K = \frac{P \cdot e}{\sum r^2}$$

$$\therefore \quad F_2 = K\, r = \frac{P \cdot e \cdot r}{\sum r^2}$$

The resultant of F_1 and F_2 act on the bolt. If 'θ' is the angle between F_1 and F_2, then the resultant F is given by

$$F = \sqrt{F_1^2 + F_2^2 + 2F_1\, F_2 \cos\theta}$$

It may be noted that farthest bolt is subjected to maximum force.

Example 3.9

A bracket plate bolted to a vertical column is loaded as shown in Fig. 3.24. If M20 bolts of grade 4.6 are used, determine the maximum value of factored load P which can be carried safely.

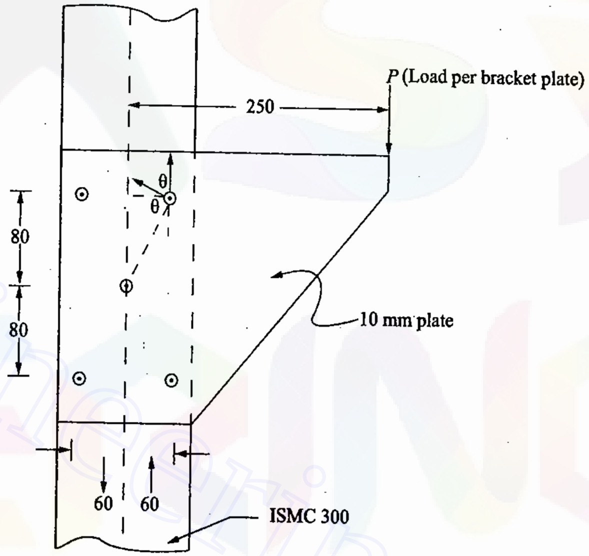

Figure 3.24

Solution:

For M20 bolts of grade 4.6, $d = 20$ $d_0 = 22$ mm $f_u = 400$ N/mm^2

For rolled steel sections, $f_u = 410$ N/mm^2

Thickness of web of ISMC 300, is

= 7.6 mm [Refer steel table].

Since this is a lap joint between bracket plate and web of ISMC 300, the bolts are in single shear.

$\therefore$ Design strength of bolts in shear

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \left(0.78 \times \frac{\pi}{4} \times 20^2 \right)$$

$$= 45272 \text{ N}$$

Strength in bearing against 7.6 mm web of ISMC 300:

K_b is the least of

$$\frac{e}{3d_0}, \ \frac{p}{3d_0} - 0.25, \ \frac{f_{ub}}{f_u}, \ 1.0$$

i.e. least of $$\frac{150-60}{3 \times 22}, \ \frac{80}{3 \times 22} - 0.25, \ \frac{400}{410}, \ 1.0$$

i.e. $$K_b = 0.96212$$

$\therefore$ Design strength of a bolt in bearing

$$= \frac{1}{1.25} \times 2.5 \, k_b \, d t f_u$$

$$= \frac{1}{1.25} \times 2.5 \times 0.96212 \times 20 \times 7.6 \times 410$$

$$= 119919 \text{ N} > 45272 \text{ N}$$

$\therefore$ Design strength of a bolt is $= 45272$ N

Force in extreme bolt:

Direct shear force $F_1 = \frac{P}{5} = 0.2P$

Centre of gravity of bolted connection is at the centre of central bolt.

For four bolts, $r = \sqrt{80^2 + 60^2} = 100$ mm.

For central bolt $r = 0$

$$\therefore \sum r^2 = 4 \times 100^2 + 0 = 4 \times 100 \times 100$$

For extreme bolt $r = 100$ mm.

$\therefore$ Force due to bending moment in extreme bolt $= \frac{P \times e \times r}{\sum r^2} = \frac{P \times 250 \times 100}{4 \times 100 \times 100} = 0.625P$

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Angle between the two forces is given by, θ where $\cos\theta = \frac{60}{100} = 0.6$

$\therefore$ Total shear force on extreme bolt

$$= \sqrt{(0.2P)^2 + (0.625P)^2 + 2 \times 0.2P \times 0.625P \times 0.6}$$

$$= P\sqrt{(0.2)^2 + (0.625)^2 + 2 \times 0.2 \times 0.625 \times 0.6}$$

$$= 0.76199 \, P$$

Equating it to strength of bolt we get

$$0.76199 \, P = 45272$$

$$\therefore \quad P = 59413 \text{ N}$$

$$P = 59.413 \text{ kN} \qquad \textbf{Answer}$$

$\therefore$ Total factored load on bracket $= 2P = 2 \times 59.413 = 118.426$ kN

3.16 DESIGN OF BEARING BOLTS SUBJECTED TO ECCENTRIC LOADING IN THE PLANE OF BOLTS

Let 'n' be the number of bolts, uniformly spaced at a distance p. The force in a bolt is proportional to its distance from the neutral axis. This maximum force in the extreme bolt should not exceed the bolt strength V.

Average force per unit depth at extreme end

$$f' = \frac{V}{p}$$

$\therefore$ Maximum force $$f = f' \frac{n}{n-1} = \frac{V}{p} \frac{n}{n-1}$$

$\therefore$ Total force above the neutral axis

$$F = \frac{1}{2} \frac{V}{p} \frac{n}{n-1} \frac{np}{2}$$

Total force below neutral axis is also equal to F and acts in the opposite direction. These two forces form a couple and resist the applied moment. Let M be the factored applied moment. Then

$$M = \text{Force} \times \text{Lever arm}$$

$$= \frac{1}{2} \frac{V}{p} \frac{n}{n-1} \frac{np}{2} \frac{2}{3} np$$

$$= \frac{Vpn^3}{6(n-1)} = \frac{Vpn^2}{6} \frac{n}{n-1}$$

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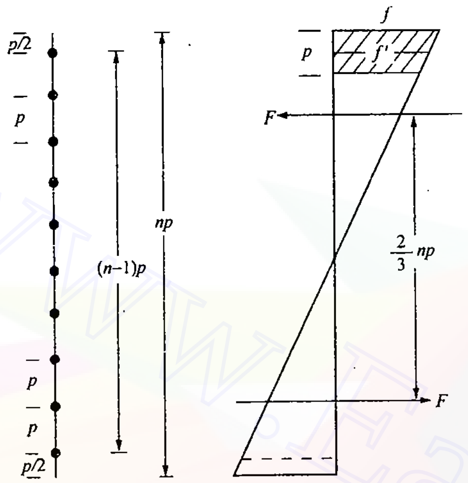

Figure 3.25

$$\therefore\ n^2 = \frac{6M}{Vp}\frac{n-1}{n}$$

$$\therefore\ n \approx \sqrt{\frac{6M}{Vp}}$$

This approximation is on safer side.

If there are two vertical lines of bolts, a value of $2V$ is used and 'n' obtained is the number of bolts required in each row. After arranging the bolts, the connection is checked for its safety.

Example 3.10

A bracket is bolted to the flange of a column as shown in Fig. 3.26, using 8 mm thick bracket plate. Using M20 bolts of grade 4.6 design the connection.

Solution:

Flange thickness of ISHB 300 @ 577 N/m is 10.6 mm. Thickness of bracket plate is 8 mm. Hence thickness of thinner member in the connection is 8 mm. For M20 bolts of grade 4.6,

$d = 20$ mm $\quad d_0 = 22$ mm $\quad f_{ub} = 400\ \text{N/mm}^2$

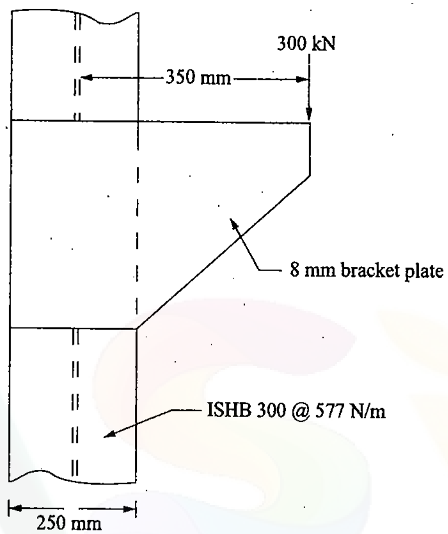

Figure 3.26 Load on each plate of bracket

For the rolled section $f_u = 410\ \text{N/mm}^2$

Bolts are in single shear.

$$\therefore\ \text{Design strength of a bolt} = \frac{1}{1.25}\times\frac{400}{\sqrt{3}}\left(0 + 0.78\times\frac{\pi}{4}\times 20^2\right)$$

i.e., $\quad V_{db} = 45272$ N

Strength of bolt in bearing:

K_b is the least of $\dfrac{e}{3d_0}$, $\dfrac{p}{3d_0} - 0.25$, $\dfrac{f_{ub}}{f_u}$ and 1.0

Adopting two rows of bolting, with edge distance of 55 mm and pitch of 50 mm ($\geq 2.5\ d_0$), K_b is the least of

$$\frac{55}{3\times 22},\ \frac{50}{3\times 22} - 0.25,\ \frac{400}{410}\ \text{and}\ 1.0$$

i.e.,
$$K_b = 0.5076$$

$$\therefore\ V_{dpb} = \frac{1}{1.25}\times 2.5\times 0.5076\times 20\times 8\times 410 = 64972\ \text{N}$$

∴ Design strength of bolt, $V = V_{db} = 45272$ N

$M = 300 \times 350$ kN-mm $= 300 \times 1000 \times 350$ N-mm.

∴ Number of bolts required per row

$$n = \sqrt{\frac{6M}{2 \times Vp}}$$

$$= \sqrt{\frac{6 \times 300 \times 1000 \times 350}{2 \times 45272 \times 50}} = 11.79$$

∴ Provide 12 bolts in each row as shown in Fig. 3.27.

Distance of extreme bolt from centre of gravity of bolts

$$r = \sqrt{70^2 + 275^2} = 283.77 \text{ mm}$$

$$\Sigma r_i^2 = 4\left[\sum_{i=1}^{6}\left(x_i^2 + y_i^2\right)\right]$$

$$= 4\left[6 \times 70^2 + 25^2 + 75^2 + 125^2 + 175^2 + 225^2 + 275^2\right]$$

$$= 832600 \text{ mm}^2$$

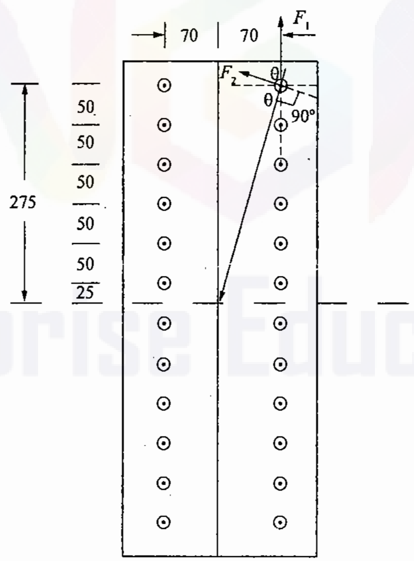

Figure 3.27

∴ Force in extreme bolt due to bending

$$= \frac{Per}{\sum r^2} = \frac{300 \times 1000 \times 350 \times 283.77}{832600}$$

$$F_2 = 35786.5 \text{ N}$$

$$\tan\theta = \frac{275}{70} \quad \therefore \theta = 75.719° \qquad \text{Hence } \cos\theta = 0.24668$$

Direct shear force $F_1 = \dfrac{300 \times 1000}{2 \times 12} = 12500$ N

∴ Resultant force on extreme bolt

$$= \sqrt{F_1^2 + F_2^2 + 2 F_1 F_2 \cos\theta}$$

$$= \sqrt{12500^2 + 35786.5^2 + 2 \times 12500 \times 35786.5 \times 0.24668}$$

$$= 40714 \text{ N} < V_{db}$$

∴ Design is safe.

Hence provide 24 M20 bolts as shown in Fig. 3.27.

3.17 TENSION CAPACITY OF BOLTS

According to IS 800-2007, clause 10.3.5, nominal tension capacity of bolt T_{nb} is given by,

$$T_{nb} = 0.9\, f_{ub}\, A_n \le f_{yb}\, A_{sb} \frac{\gamma_{mb}}{\gamma_{mo}}$$

and design capacity T_{db} is given by

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$

$$\therefore \quad T_{db} = \frac{0.9\, f_{ub}\, A_n}{\gamma_{mb}} \le \frac{f_{yb}\, A_{sb}}{\gamma_{mo}}$$

where

f_{ub} = ultimate tensile stress of bolt

f_{yb} = yield stress of the bolt

A_n = net area of the rest of bolt and

A_{sb} = shank area of the bolt.

For ordinary (bearing-bolt) bolt of grade 4.6,

$$f_{ub} = 400 \text{ N/mm}^2 \quad f_{yb} = 240 \text{ N/mm}^2$$

$$A_n = 0.78\frac{\pi}{4}d^2 \quad A_{sb} = \frac{\pi}{4}d^2$$

$$\gamma_{mb} = 1.25 \quad \gamma_{mo} = 1.1$$

Hence,

$$T_{db} = \frac{0.9\times 400\times 0.78\left(\frac{\pi}{4}\right)d^2}{1.25} \le \frac{240\times\left(\frac{\pi}{4}\right)d^2}{1.1}$$

$$= 176.432\, d^2 \le 171.360\, d^2$$

$$\therefore T_{db} = 171.360\, d^2$$

Thus yield stress criteria governs the tension capacity i.e.

$$T_{db} = \frac{240\times\frac{\pi}{4}d^2}{1.1}$$

If T_b is factored tensile force, the design criteria is $T_b \le T_{db}$.

3.18 DESIGN CRITERIA FOR BOLT SUBJECTED TO COMBINED SHEAR AND TENSION

According to IS 800-2007, clause 10.3.6, a bolt required to resist both design shear force V_{sb} and design tensile force T_b at the same time shall satisfy

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

where V_{db} – design shear strength and T_{db} – design tensile capacity of bolts.

3.19 DESIGN OF BEARING BOLTS SUBJECTED TO ECCENTRIC LOADING CAUSING MOMENT IN THE PLANE PERPENDICULAR TO THE PLANE OF GROUP OF BOLTS

This type of connection is shown in Fig. 3.23. Referring to Fig. 3.28, let P be factored load at an eccentricity 'e'. Then the section is subjected to a direct shear force P and moment $M = P \times e$.

If there are 'n' number of bolts in the connection, direct design shear force on each bolt is given by

$$V_{sb} = \frac{P}{n}$$

The moment causes tension in top side and compression in the bottom side. On tension side, only bolts resist load but on compression side entire contact zone between the columns and the connecting angle resist the load. Hence the neutral axis will be much below in these connections. It is assumed to lie at a height of $\frac{1}{7}$th of the depth of the bracket, measured from the bottom edge of the angle.

The variation of the force is as shown in Fig. 3.28(c).

The tensile force in a bolt T_{bi} is proportional to its distance y_i from the line of rotation.

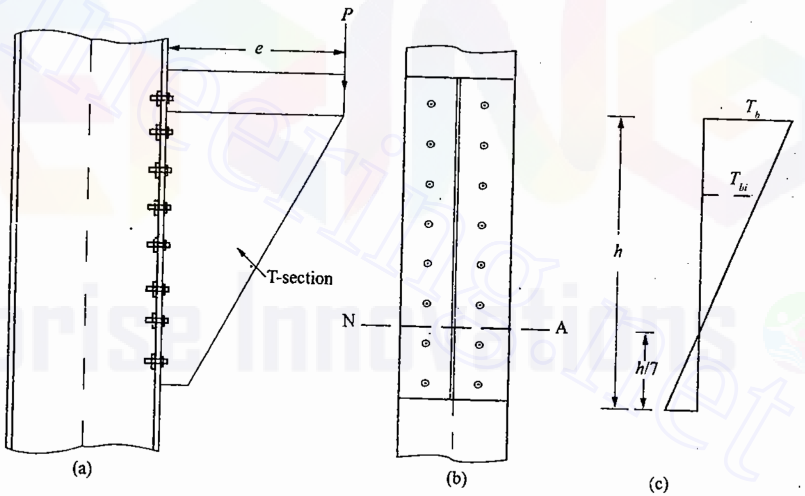

Figure 3.28

$$T_{bi} \propto y_i$$

$$= k y_i, \text{ where } k \text{ is constant.}$$

$$\therefore \quad k = \frac{T_{bi}}{y_i}$$

Total moment of resistance M' provided by bolts in tension.

$$M' = \sum T_{bi}\, y_i = \sum k\, y_i^2$$

$$= k \sum y_i^2 = \frac{T_{bi}}{y_i} \sum y_i^2$$

or

$$T_{bi} = \frac{M' y_i}{\sum y_i^2}$$

$\therefore$ Total tensile force in bolts

$$T = \sum T_{bi} = \frac{M' \sum y_i}{\sum y_i^2}$$

For equilibrium,

total tensile force = total compressive force

$$T = C = \frac{M' \sum y_i}{\sum y_i^2}$$

Taking moment about neutral axis,

$$M = M' + C \frac{2}{3} \frac{h}{7}$$

$$= M' \left[1 + \frac{2h}{21} \frac{\sum y_i}{\sum y_i^2} \right]$$

or

$$M' = \frac{M}{\left[1 + \frac{2h}{21} \frac{\sum y_i}{\sum y_i^2} \right]}$$

$\therefore$ Tensile force T_{db} in extreme bolt can be found.

This equation gives the moment resisted by the bolts in tension from which the maximum tensile force in the extreme bolt T_b can be calculated. Then the design requirement is

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

Steps to be followed in the design:

Step 1: Select nominal diameter 'd' of bolts.

Step 2: Adopt a pitch (p) of 2.5d to 3d for bolts.

Step 3: Bolts are to be provided in two vertical rows. Number of bolts necessary in each row is computed from the expression,

$$n = \sqrt{\frac{6M}{(2V)\,p}}$$

where M is the moment on the joint and V is the design strength of bolt.

Step 4: Find the direct shear and tensile forces acting on the extreme bolt. If it is HSFG bolted connection add prying force (Ref. Fig. 3.23) to direct tension. Check whether the interaction formula is satisfied.

Example 3.11

Design a suitable bolted bracket connection of a ISHT-75 section attached to the flange of a ISHB 300 at 577 N/m to carry a vertical factored load of 600 kN at an eccentricity of 60 mm. Use M24 bolts of grade 4.6. [Ref. Fig. 3.28]

Solution:

For M24 bolts of grade 4.6,

$d = 24$ mm $\quad d_0 = 26$ mm $\quad f_{ub} = 400$ N/mm^2 $\quad f_{yb} = 240$ N/mm^2

For rolled steel section, $f_u = 410$ N/mm^2

Thickness of flange of ISHT 75 (from steel table) = 9 mm.

For ISHB 300 @ 577 N/m, thickness of flange = 10.6 mm

$\therefore$ Thickness of thinner member = 9 mm.

Design strength of bolts in single shear $= \dfrac{1}{1.25} \dfrac{400}{\sqrt{3}} \left(0 + 0.78 \times \dfrac{\pi}{4} \times 24^2\right)$

$= 65192$ N

Design strength of bolts in bearing:

Minimum edge distance $e = 1.5 \times d_0 = 1.5 \times 26 = 39$ mm

Minimum pitch $p = 2.5\ d = 2.5 \times 24 = 60$ mm

Providing $e = 40$ mm and $p = 60$ mm,

K_b is minimum of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}$ and 1.0

i.e., minimum of $\frac{40}{3 \times 26}, \frac{60}{3 \times 26} - 0.25, \frac{400}{410}$ and 1.0

$\therefore\ K_b = 0.519$

$\therefore$ Design strength of bolts in bearing against 9 mm thick web of Tee section

$$= \frac{1}{1.25} \times 2.5 \times k_b\, dt\, f_u$$

$$= \frac{1}{1.25} \times 2.5 \times 0.519 \times 24 \times 9 \times 410$$

$$= 91925 \text{ N} > 65192 \text{ N}$$

$\therefore$ Design strength of bolts $V = V_{db} = 65192$ N

Design tension capacity of bolts

$$T_{db} = \frac{0.90 f_{ub} A_n}{\gamma_m} < \frac{f_{yb} A_{sb}}{\gamma_{mo}}$$

$$= \frac{0.90 \times 400}{1.25} \times 0.78 \times \frac{\pi}{4} \times 24^2 < \frac{240 \times \frac{\pi}{4} \times 24^2}{1.1}$$

101624 N< 98703 N

$\therefore T_{db} = 98703$ N

Using two rows of bolting, approximate number of bolts required in each row

$$n = \sqrt{\frac{6}{(2V)p}} \sqrt{\frac{6 \times 600 \times 1000 \times 60}{2 \times 65192 \times 70}} = 5.7$$

Provide 6 bolts in each row as shown in Fig. 3.29.

$\therefore\ h = 40 + 60 \times 5 = 340$ mm.

$$\frac{h}{7} = \frac{340}{7} = 48.57 \text{ mm}$$

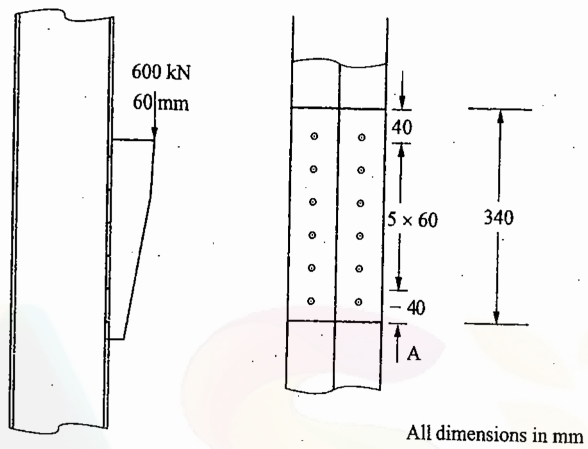

Figure 3.29 (Example 3.11)

i.e., neutral axis lies between 1st and 2nd bolts.

$\therefore\ y$ of second bolt $= (40 + 60) - 48.57 = 51.43$ mm

Bolt No.	2	3	4	5	6
y in mm	51.43	111.43	171.43	231.43	291.43

Since there are two rows of bolts $\sum y = 2 \times 857.15$ mm

$$\sum y^2 = 2 \times 182941 \text{ mm}^2$$

$\therefore$ Total moment resisted by bolts in tension

$$M' = \frac{M}{1 + \frac{2h}{21} \frac{\sum y_i}{\sum y_i^2}} = \frac{600 \times 1000 \times 60}{1 + \frac{2 \times 340}{21} \times \frac{2 \times 857.15}{2 \times 182941}}$$

$$= 31.2577 \times 10^6 \text{ N-mm}$$

$\therefore$ Tensile force in extreme bolt due to bending moment

$$T_b = \frac{M' y_i}{\sum y_i^2} = \frac{31.2577 \times 10^6}{2 \times 18941} \times 291.43 = 24897 \text{ N}$$

Direct shear force $V_{sb} = \frac{600 \times 1000}{2 \times 6} = 50000$ N

Check by interaction formula $= \left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2$

$$= \left(\frac{5000}{65192}\right)^2 + \left(\frac{24897}{98703}\right)^2$$

$$= 0.652 < 1.0$$

Hence the bolts are safe. Provide bolts as shown in Fig. 3.29.

3.20 SHEAR CAPACITY OF HSFG BOLTS

As stated in Art. 3.2, these are the bolts made of high tensile steel which are pretensioned and then provided with nuts. The nuts are clamped also. Hence resistance to shear force is mainly by friction.

There are two types of HSFG bolts. They are parallel shank and waisted shank type. Parallel shank type HSFG bolts are designed for no-slip at serviceability loads. Hence they slip at higher loads and slip into bearing at ultimate load. Such bolts should be checked for their bearing strength at ultimate load. Waisted shank HSFG bolts are designed for no slip even at ultimate load and hence there is no need to check for their bearing strength.

IS 800-2007 (clause 10.4) recommends use of the following expression for finding nominal shear capacity of HSFG (parallel shank or waisted shank) bolts:

$$V_{nsf} = \mu_f n_e K_h F_0$$

where,

μ_f = coefficient of friction (called slip factor) as specified in Table 3.1.

n_e = number of effective interfaces offering frictional resistance to the slip.

[*Note:* $n_e = 1$ for lap joints and 2 for double cover butt joints]

K_h = 1.0 for fasteners in clearance holes

= 0.85 for fasteners in oversized and short slotted holes and for long slotted holes loaded perpendicular to the slot.

= 0.70 for fasteners in long slotted holes loaded parallel to the slot.

F_0 = Minimum bolt tension at installation and may be taken as $A_{nb} f_0$

A_{nb} = net area of the bolt at threads $\left(= 0.78 \frac{\pi}{4} d^2\right)$

f_0 = proof stress = $0.70 f_{ub}$

Table 3.1 Typical average value for coefficient of friction (μ_f) [Table 20 in IS 800-2007]

Sl. No.	Treatment of Surface	μ_f
1	Surface not treated	0.20
2	Surface blasted with shot or grit with any loose rust removed, no pitting	0.50
3	Surface blasted with shot or grit and hot-dip galvanized	0.7
4	Surface blasted with shot or grit and spray-metallized with zinc (thickness 50–70 μm)	0.25
5	Surfaces blasted with shot or grit and painted with ethylzinc silicate coat (thickness 30–60 μm)	0.30
6	Sand blasted surface, after light rusting	0.52
7	Surface blasted with shot or grit and painted with ethylzinc silicate coat (thickness 60–80 μm)	0.30
8	Surface blasted with shot or grit and painted with alkalizinc silicate coat (thickness 60–80 μm)	0.30
9	Surface blasted with shot or grit and spray metalled with aluminium (thickness > 50 μm)	0.50
10	Clean mill scale	0.33
11	Sand blasted surface	0.48
12	Red lead painted surface	0.1

The slip resistance should be taken as

$$V_{sf} = \frac{V_{nsf}}{\gamma_{mf}}$$

where γ_{mf} = 1.10, if the slip resistance is designed at service load (parallel shank HSFG)

= 1.25, if the slip resistance is designed at ultimate load (waisted shank HSFG).

It may be noted that the reduction factors specified (Art. 3.11) for bearing bolts hold good for HSFG bolts also.

For commonly used HSFG bolts (grade 8.8), yield stress f_{yb} = 640 N/mm² and ultimate stress f_{ub} = 800 N/mm².

Example 3.12

Determine the shear capacity of bolts used in connecting two plates as shown in Fig. 3.30, if

(i) Slip resistance is designated at service load

(ii) Slip resistance is designated at ultimate load

Given:

(1) HSFG bolts of grade 8.8 are used.
(2) Fasteners are in clearance holes.
(3) Coefficient of friction = 0.3.

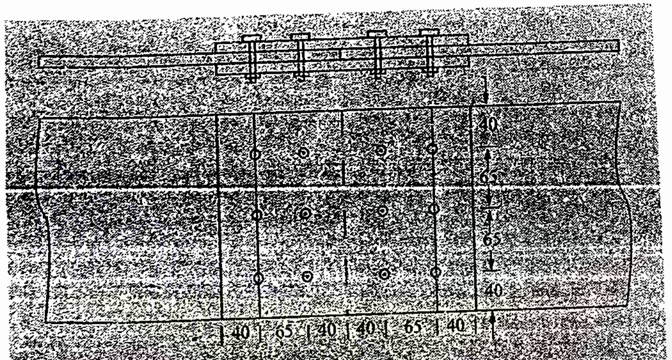

Figure 3.30

Solution:

For HSFG bolts of grade 8.8,

$$f_{ub} = 800 \text{ N/mm}^2$$

For fasteners in clearance holes $K_h = 1.0$

Coefficient of friction $\mu_f = 0.3$ (given)

∴ Nominal shear capacity of a bolt

$$V_{nsf} = \mu_f n_c K_h F_0$$

where $F_0 = 0.7 f_{ub} A_{nb}$

$$= 0.7 \times 800 \times 0.78 \times \frac{\pi}{4} \times 20^2 = 137225 \text{ N}$$

$n_e = 2$, since it is double cover butt joint

$$\therefore \quad V_{nsf} = 0.3 \times 2 \times 1.0 \times 137225$$

$$= 82335 \text{ N}$$

(i) Design capacity of one bolt, if slip resistance is designated at service load

$$= \frac{82335}{1.1} = 74850 \text{ N}$$

∴ Design capacity of joint = 6 × 74850, since 6 bolts are used

$$= 449099 \text{ N}$$

$$= 449.099 \text{ kN}$$

(ii) Design capacity of one bolt, if the slip resistance is designated at ultimate load

$$= \frac{82335}{1.25} = 65868 \text{ N.}$$

∴ Design capacity of joint = 6 × 65868

$$= 395208 \text{ N}$$

$$= 395.208 \text{ kN}$$

In case (i), bearing strength at ultimate load should be checked. If it is low that will be the governing factor.

3.21 TENSION RESISTANCE OF HSFG BOLTS

The expression for nominal tension strength of HSFG bolts is also same as that for bearing bolts. i.e,

$$T_{nf} = 0.9 f_{ub} A_n \leq f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{mo}}$$

and hence $$T_{df} = \frac{0.9 f_{ub} A_n}{\gamma_{mb}} \leq \frac{f_{yb} A_{sb}}{\gamma_{mo}}$$

where A_n = net tensile area as specified in various parts of IS 1367, it may be taken as the area at the root of the thread $\approx 0.78 \frac{\pi d^2}{4}$

A_{sb} = shank area.

$\gamma_{mb} = 1.25$, $\gamma_{mo} = 1.1$

f_{ub} for bolts of grade 8.8 is 800 MPa and $f_{yb} = 640$ MPa.

3.22 INTERACTION FORMULA FOR COMBINED SHEAR AND TENSION

If bolts are under combined action of shear and axial tension, the interaction formula to be satisfied is

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

3.23 PRYING FORCES

In the design of HSFG bolts subjected to tensile forces, an additional force, called as prying force Q is to be considered. These additional forces are mainly due to flexibility of connected plates. Consider the connection of a T-section to a plate as shown in Fig. 3.31, subject to tensile force $2T_e$.

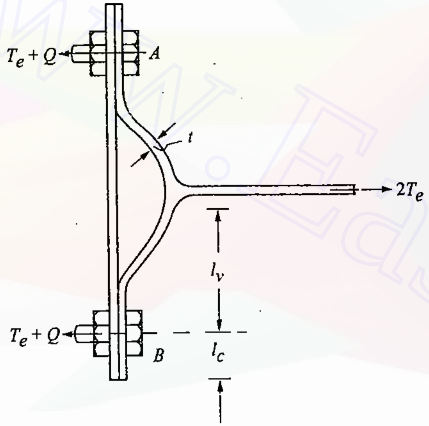

Figure 3.31

As tensile force acts, the flange of T-section bends in the middle portion and presses connecting plate near bolts. It gives rise to additional contact forces known as prying forces. During late 80s and early 90s lot of research works were published regarding assessing prying forces. IS 800-2007 has accepted the following expression

$$Q = \frac{l_v}{2l_c}\left(T_e - \frac{\beta\eta f_0 b_e t^4}{27 l_c l_v^2}\right)$$

where,

Q = prying force

$2T_e$ = total applied tensile force

l_v = distance from the bolt centre line to the toe of the fillet weld or to half the root radius for a rolled section.

l_c = distance between prying forces and bolt centre line and is the minimum of either the end distance or the value given by:

$$l_c = 1.1t\sqrt{\frac{\beta f_0}{f_y}}$$

β = 2 for non-pretensioned bolts and 1 for pretensioned bolts

η = 1.5

b_e = effective width of flange per pair of bolts.

f_0 = proof stress in consistent units

t = thickness of end plate.

Note that prying forces do not develope in case of ordinary bolts, since when bolt failure takes place contact between the two connecting plates is lost (Ref. Fig. 3.32).

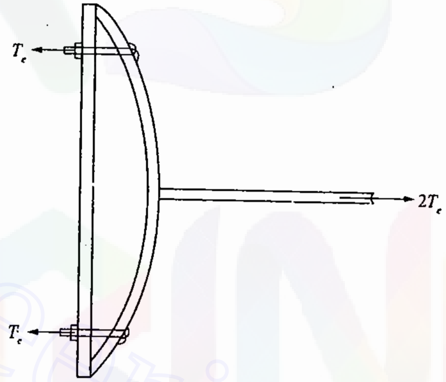

Figure 3.32

Example 3.13

The joint shown in Fig. 3.33, has to carry a factored load of 180 kN. End plate used is of size 160 mm × 140 mm × 16 mm. The bolts used are M20 HSFG of grade 8.8. Check whether the design is safe.

Solution:

Assuming 8 mm weld and edge distance 40 mm,

$$l_v = \frac{160}{2} - 8 - 8 - 40 = 24 \text{ mm}$$

$$l_c = 1.1t\sqrt{\frac{\beta f_0}{f_y}}$$

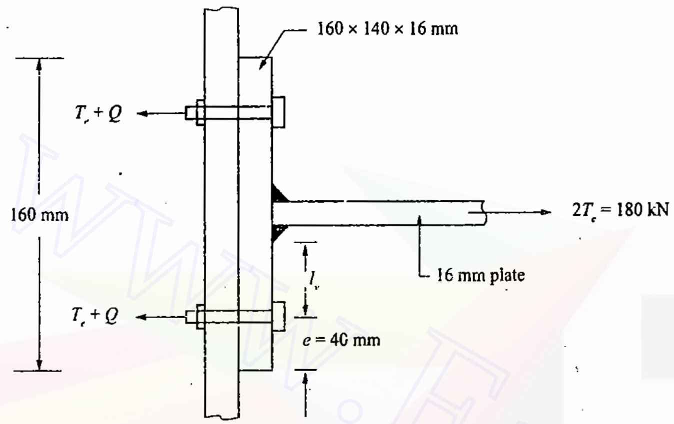

Figure 3.33

For plates, $f_0 = 0.7 f_u$, $f_u = 410$ MPa and $f_y = 250$ MPa, $f_{ub} = 800$ MPa

$$\therefore \quad l_c = 1.1 \times 16 \sqrt{\frac{1 \times 0.7 \times 800}{250}} = 26.34 < \text{edge distance}$$

$\therefore \quad l_c = 26.34$ mm

Prying force is given by,

$$Q = \frac{l_v}{2 l_c}\left[T_e - \frac{\beta \eta f_0 b_e t^4}{27 l_c l_v^2}\right]$$

Now, $\beta = 1.0$, for pretensioned bolts.

$\eta = 1.5$

$f_0 = 0.7 \times 800 = 560$ MPa

$b_e = 140$ mm, $t = 16$ mm.

$$\therefore \quad Q = \frac{24}{2 \times 26.34}\left[90000 - \frac{1 \times 1.5 \times 560 \times 140 \times 16^4}{27 \times 26.34 \times 24^2}\right]$$

$$= 32430.9 \text{ N}$$

$\therefore$ Tension to be resisted by the bolt

$$T = T + Q = 90000 + 32430.9 = 122430.9 \text{ N}$$

$$\text{Tension capacity of the bolt} = \frac{0.9 f_{ub} A_{nb}}{1.25}$$

$$= \frac{0.9 \times 800 \times 0.78 \times \frac{\pi}{4} \times 20^2}{1.25}$$

$$= 141145 \text{ N} > 122430.9 \text{ N}$$

Hence the design is safe.

Example 3.14

A bracket is made by welding a 20 mm thick, 150 mm wide plate to a 12 mm thick plate as shown in Fig. 3.34, the thickness of fillet weld being 8 mm. This is to be connected to the flange of the column ISHB 300 @ 577 N/m. Using M24 HSFG bolts of grade 8.8 design the bolted connection, assuming coefficient of friction, $\mu = 0.48$.

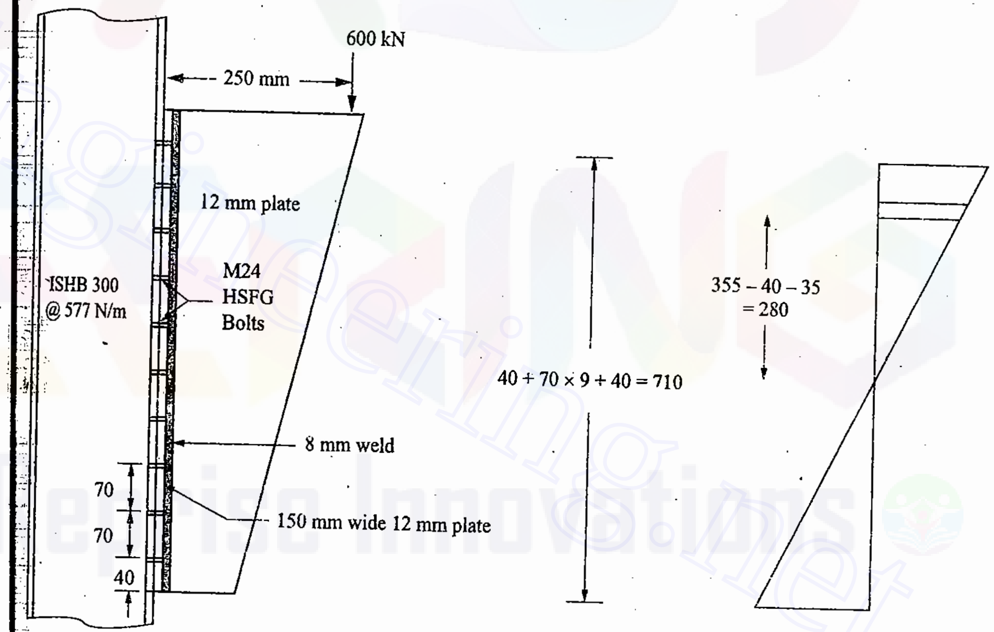

Figure 3.34

Solution:

For M24, HSFG bolts of grade 8.8, $d = 24$ mm, $f_{ub} = 800$ MPa, $f_y = 640$ MPa

Design shear strength of bolts

$$V_{dsf} = \frac{1}{1.25}\mu_f n_e k_n F_0$$

$\mu_f = 0.48, n_e = 1, k_n = 1.0$ for fasteners in clearance

$$F_0 = A_{nb} f_0 = 0.78 \times \frac{\pi}{4} \times 24^2 \times 0.7 \times 800$$

$$= 197604 \text{ N}$$

$$\therefore \quad V_{dsf} = \frac{1}{1.25}\times 0.48\times 1\times 1.0\times 197604 = 75880 \text{ N}$$

Since there are two rows of bolts in the connection, number of bolts required per row when $p = 70$ mm and taking $V = V_{dsf}$, we get

$$n = \sqrt{\frac{6M}{(2V)\times p}}$$

$$= \sqrt{\frac{6\times 600\times 1000\times 250}{2\times 75880\times 70}} = 9.2$$

Provide 10 bolts in each row with edge distance 40 mm as shown in Fig. 3.34.

Tensile capacity of bolts $= \frac{1}{1.25} \times 0.9 \times f_{ub} \times A_{nb} = \frac{1}{1.25} \times 0.9 \times 800 \times 0.78 \times \frac{\pi}{4} \times 24^2$

$$= 203250$$

When there is no load, the bracket is held on to the column by compression developed due to the bolt tension. This phenomenon continues even after the load is applied. Hence the interface of area 150×710 mm may be considered as a plane in the monolithic beam. The stress diagram is as shown in Fig 3.34.

$\therefore$ Maximum bending stress $= \frac{6M}{bd^2} = \frac{6 \times 600 \times 10^3 \times 250}{150 \times 710^2}$

$$= 11.9 \text{ N/mm}^2$$

Bending stress at $40 + \frac{70}{2} = 75$ mm from top fibre

$$= 11.9 \times \frac{(355 - 75)}{355} = 9.39 \text{ N/mm}^2$$

$$\text{Average stress} = \frac{11.9 + 9.39}{2} = 10.64 \text{ N/mm}^2$$

This average bending stress could be considered as tension in the bolt.

$\therefore$ Tension in extreme two bolt $= 10.64 \times 150 \times 75$

$$= 119744 \text{ N}$$

$$\therefore \quad T_e = \frac{119744}{2} = 59872 \text{ N}$$

Prying forces:

Plate width = 150 mm and thickness = 12 mm

$$l_v = \frac{150}{2} - 6 - 8 - 40 = 21 \text{ mm}$$

For the connecting plate, $f_u = 410$ MPa, $f_y = 250$ MPa

$$\therefore \quad l_e = 1.1\times 12\times\sqrt{\frac{1\times 0.7\times 410}{250}} = 14.14 < \text{edge distance}$$

$$\therefore \quad l_e = 14.14 \text{ mm}$$

$\beta = 1.0$, for pretensioned bolts

$b_e = 150$ mm, $f_0 = 0.7$, $f_{ub} = 0.7 \times 800 = 560$ MPa

$t = 12$ mm

$\therefore$ Prying force Q is given by

$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_0 b_e t^4}{27 l_e l_v^2}\right]$$

$$= \frac{21}{2\times 14.14}\left[59872 - \frac{1\times 1.5\times 560\times 150\times 12^4}{27\times 14.14\times 21^2}\right]$$

$$= 44354 \text{ N.}$$

$\therefore$ Total tensile force in the bolt

$$T_f = 59872 + 44354 \text{ N}$$

$$= 104226 \text{ N}$$

Tension capacity $T_{df} = \frac{1}{1.25} \times 0.9 f_{ub} A_{nb} = \frac{1}{1.25} \times 0.9 \times 800 \times 0.78 \times \frac{\pi}{4} \times 24^2$

$$= 203249 \text{ N}$$

$$\text{Direct shear in the bolt} = \frac{600\times 1000}{2\times 10} = 30000 \text{ N}$$

$$\therefore \left(\frac{V_{st}}{V_{dsf}}\right)^2 + \left(\frac{T_f}{T_{dsf}}\right)^2 = \left(\frac{30000}{75880}\right)^2 + \left(\frac{104226}{203249}\right)^2$$

$$= 0.419 \leq 1.0$$

$\therefore$ The design is safe.

* * *

Questions

1. Write short notes on
   (a) Riveted connection.
   (b) HSFG bolts.
2. Distinguish between
   (a) Black bolts and turned bolts.
   (b) Bearing bolts and friction grip bolts.
3. Discuss the advantages and disadvantages of
   (a) Riveted connection and bolted connection.
   (b) Bearing bolts and HSFG bolts.
   (c) Black bolts and turned (finished) bolts.
4. Explain the following terms:
   (a) Pitch of Bolts
   (b) Gauge Distance
   (c) Edge Distance
   (d) Staggered Distance
   (e) Tacking Bolts.
5. List the assumptions made in the design of bearing bolts.
6. Two plates 16 mm are to be joined using M20 bolts of grade 4.6 in
   (a) Lap joint.
   (b) Butt joint using 10 mm cover plates.
   Determine the bolt value.
7. If the joint specified in Question 6 is made with M20 HSFG of grade 8.8, find the bolt value. Take coefficient of friction = 0.48.
8. An angle section 8 mm thick carrying 120 kN factored load is to be connected to a gusset plate (lap joint) using M20 bolts of grade 4.6. Find the number of bolts required and sketch the connection details.
9. The plates of a boiler are 10 mm thick, connected by M16 bolts of grade 4.6 at a spacing of 50 mm. If it is lap joint, determine the efficiency of the connection.

10. Find the maximum force which can be transferred through the double covered butt joint shown in Fig. 3.19. Find the efficiency of the joint also. M24 bolts of grade 4.6 and Fe 410 plates are used.
11. Two cover plates, 10 mm and 20 mm thick are to be connected by double cover butt joint using 8 mm cover plates (similar to refer Fig. 3.20). Find the strength of the joint, if M16 bolts of grade 4.6 and Fe 415 plates are used.
12. Find the safe load P carried by the joint shown in Fig. 3.35. M20 bolts of grade 4.6 are provided at a pitch of 80 mm. The thickness of the flange is 6.1 mm and that of the bracket plate is 8 mm.
    Note: Safe load $= \dfrac{\text{Factored load}}{1.5}$

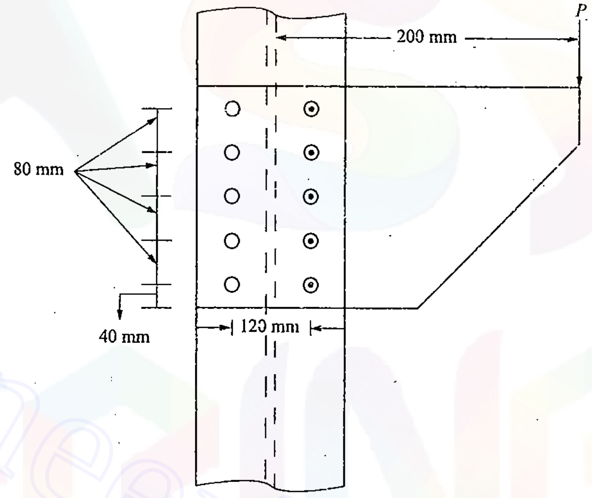

Figure 3.35

13. The dimensions of a plate, bracket bolted to the face of the stanchion are shown in Fig. 3.36. Determine the resultant stress in the most heavily loaded bolt of the group, allowing both tensional and direct shear. The bolts used are M20 of grade 4.6.
    [*Hint:* Moment = 50 cos 45 × 250 kN/mm; Horizontal component 50 cos 45° kN causes horizontal shear while vertical component 50 cos 45° causes vertical shear. Finally resultant of 3 forces are to be found.]
14. Design the bolted connection shown in Fig. 3.28, if P = 400 kN and e = 250 mm. Use M20 bolts of grade 4.6.
15. Design the bolted connection specified in Q.No. 14 using M20 HSFG bolts of grade 8.8.

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4

WELDED CONNECTIONS

Welding consists of joining two pieces of metal by establishing a metallurgical bond between them. The elements to be connected are brought closer and the metal is melted by means of electric arc or oxy-acetylene flame alongwith weld rod which adds metal to the joint. After cooling the bond is established between the two elements.

In this chapter advantages and disadvantages of welded connections are discussed and different types of welded connections are explained. After briefly explaining IS 800-2007 specifications for welded connections the method of designing welded connections and bracket connections is illustrated with examples.

4.1 ADVANTAGES AND DISADVANTAGES OF WELDED CONNECTIONS

The following are the advantages of welded connections:

1. Due to the absence of gusset plates, connecting angles etc., welded structures are *lighter*.
2. The absence of making holes for fasteners, makes welding process *quicker*.
3. Welding is *more adaptable* than bolting or riveting. For example, even circular tubes can be easily connected by welding.
4. It is possible to achieve *100 percent efficiency* in the joint whereas in bolted connection it can reach a maximum of 70–80 percent only.
5. *Noise produced* in welding process is relatively less.
6. Welded connections have *good aesthetic appearance*.
7. Welded connection is *airtight and watertight*. Hence there is less danger of corrosion of steel structures and welded connection is preferred for making water tanks.
8. Welded joints are rigid.
9. There is no problem of *mismatching of holes* in welded connections whereas in bolted connections mismatching of bolt holes creates considerable problem.
10. *Alterations* in connections can be easily made in the design of welded connections.

The following are the disadvantages of welded connection:

1. Due to uneven heating and cooling, members are *likely to distort* in the process of welding.
2. There is a greater possibility of *brittle fracture* in welding.
3. A welded joint fails earlier than a bolted joint, if the structure is *under fatigue* stresses.

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4. The *inspection* of welded joints is *difficult* and *expensive*. It needs non-destructive testing.
5. Highly *skilled person* is required for welding.
6. *Proper welding in field* conditions is difficult.
7. Welded joints are *over rigid*.

4.2 TYPES OF WELDED JOINTS

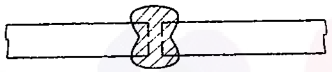
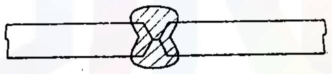
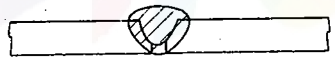
There are three types of welded joints:

1. Butt weld
2. Fillet weld
3. Slot weld and Plug weld.

4.2.1 Butt Weld

Butt weld is also known as groove weld. Depending upon the shape of the groove made for welding butt welds are classified as shown in Table 4.1.

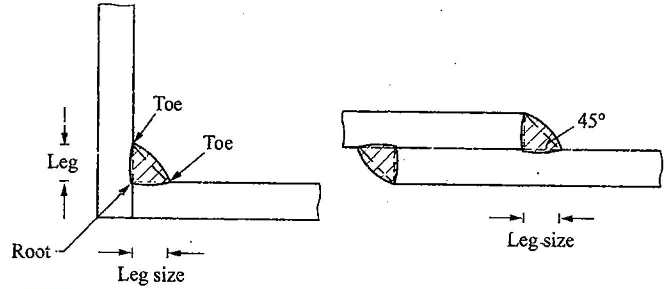
Table 4.1 Types of butt weld

Sl. No.	Type of Butt Weld	Sketch
(a)	Square butt weld, on one side	
(b)	Square butt weld, both sides	
(c)	Single V butt joint	
(d)	Double V-butt joint	
(e)	Single U butt joint	
(f)	Single J-butt joint	
(g)	Single bevel butt joint	

Note: Similarly there can be double U, double J and double bevel butt joints.

4.2.2 Fillet Weld

Fillet weld is a weld of approximately triangular cross-section joining two surfaces approximately at right angles to each other in lap joint, tee joint or corner joint. Figure 4.1 shows typical fillet welds.

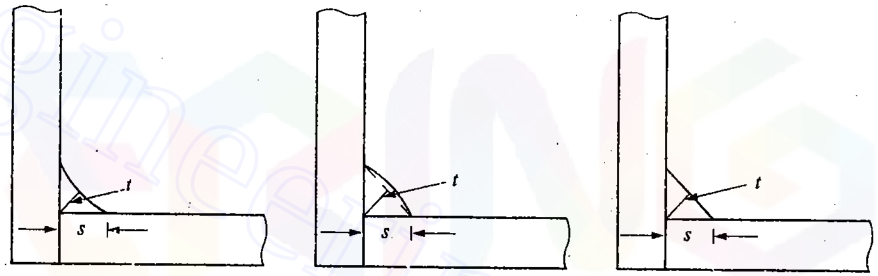


(a) (b)

Figure 4.1 Typical fillet welds.

When the cross-section of fillet weld is isoceles triangle with face at 45°, it is known as a *standard fillet weld.* In special circumstances 60° and 30° angles are also used.

A fillet weld is known as concave fillet weld, convex fillet weld or as mitre fillet weld depending upon the shape of weld face, [Refer Fig. 4.2]



(a) Concave (b) Convex (c) Mitre

s - size *t* - throat thickness

Figure 4.2 Types of fillet welds.

4.2.3 Slot Weld and Plug Weld

Figure 4.3 shows a typical slot weld in which a plate with circular hole is kept with another plate to be joined and then fillet welding is made along the periphery of the hole.

Figure 4.4 shows typical plug welds in which small holes are made in one plate and is kept over another plate to be connected and then the entire hole is filled with filler material.

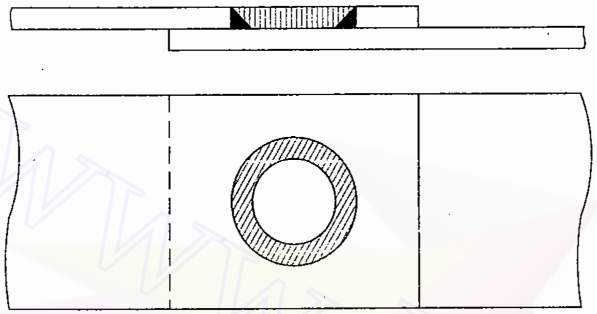


Figure 4.3 Slot weld.

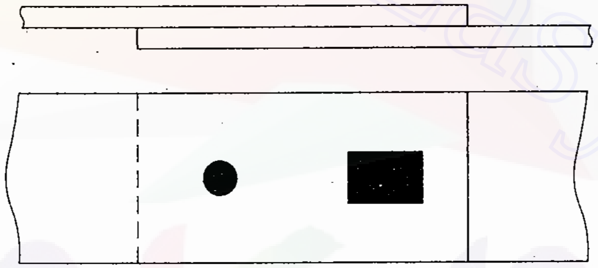


Figure 4.4 Plug welds.

4.3 IMPORTANT SPECIFICATIONS FOR WELDING

Requirements of welds and welding shall conform to IS 816 and IS 9595, as appropriate. Some of important specifications regarding butt weld, fillet weld and plug and slot welds as per IS 800-2007 are listed below:

4.3.1 Butt Weld

1. The size of butt weld shall be specified by the effective throat thickness. In case of a complete penetration butt weld it shall be taken as thickness of the thinner part joined. Double U, double V, double J and double bevel butt welds may be generally regarded as complete penetration butt welds.

   The effective throat thickness in case of incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement. In the absence of actual data it may be taken as 5/8th of thickness of thinner material (IS 800-1969).

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2. The effective length of butt weld shall be taken as the length of full size weld.
3. The minimum length of butt weld shall be four times the size of the weld.
4. If intermittent butt welding is used, it shall have an effective length of not less than four times the weld size and space between the two welds shall not be more than 16 times the thickness of the thinner part joined.

4.3.2 Fillet Weld

1. Size of fillet weld:

   (a) The size of normal fillet weld shall be taken as the minimum weld leg size.

   (b) For deep penetration welds with penetration not less than 2.4 mm, size of weld is minimum leg size + 2.4 mm.

   (c) For fillet welds made by semi automatic or automatic processes with deep penetration more than 2.4 mm, if purchaser and contractor agree,

$$s = \text{minimum leg size} + \text{actual penetration}$$

2. Minimum size of fillet weld specified is 3 mm. To avoid the risk of cracking in the absence of preheating the minimum size specified are

For less than 10 mm plate	3 mm
For 10 to 20 mm plate	5 mm
For 20 to 32 mm plate	6 mm
For 32 to 50 mm plate	8 mm

3. Effective throat thickness: It shall not be less than 3 mm and shall not generally exceed $0.7t$ (or t under special circumstances), where t is the thickness of the thinner plate of the elements being welded.

   If the faces of plates being welded are inclined to each other, the effective throat thickness shall be taken as K times the fillet size where K is as given in table below:

Angle between fusion faces	60°–90°	91°–100°	101°–106°	107°–113°	114°–120°
Constant K	0.70	0.65	0.60	0.55	0.5

4. Effective length: The effective length of the weld is the length of the weld for which specified size and throat thickness exist. In drawings only effective length is shown. While welding length made is equal to effective length plus twice the size of the weld. Effective length should not be less than four times the size of the weld.
5. Lap joint: The minimum lap should be four times the thickness of thinner part joined or 40 mm whichever is more. The length of weld along either edge should not be less than the transverse spacing of welds.

6. Intermittent welds: Length shall not be less than 4 times the weld size or 40 mm whichever is more. The minimum clear spacing of intermittent weld shall be $12t$ for compression joints and $16t$ for tensile joints, where t is the thickness of thinner plate joined. The intermittent welds shall not be used in positions subject to dynamic, repetitive and alternating stresses.

4.3.3 Plug Welds

The effective area of a plug weld shall be considered the nominal area of the hole.

4.4 DESIGN STRESSES IN WELDS

Butt Welds

Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal.

Fillet Weld, Slot or Plug Welds

Design strength shall be based on its throat area and shall be given by

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

where, $f_{wn} = \dfrac{f_u}{\sqrt{3}}$

f_u = smaller of the ultimate stress of the weld or of the parent metal.

γ_{mw} = 1.25 for shop welds
= 1.5 for field welds.

The following provisions are made in the code for the fillet welds applied to the edge of a plate or section:

1. If a fillet weld is to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness (Ref. Fig. 4.5)

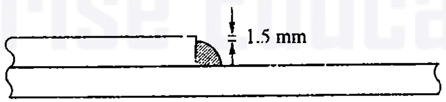

Figure 4.5

2. If fillet weld is to the rounded toe of a rolled section, the specified size of the weld should generally not exceed ¾th of the thickness of the section at the toe (Fig. 4.6).

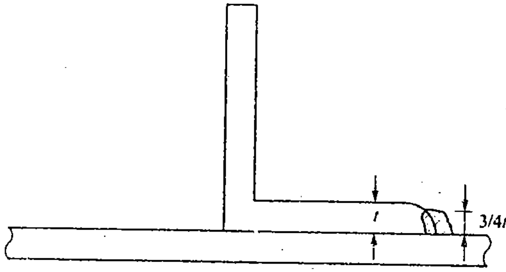

Figure 4.6

3. In members subject to dynamic loading, the fillet weld shall be of full size (Fig. 4.7) with its leg length equal to the thickness of plate.

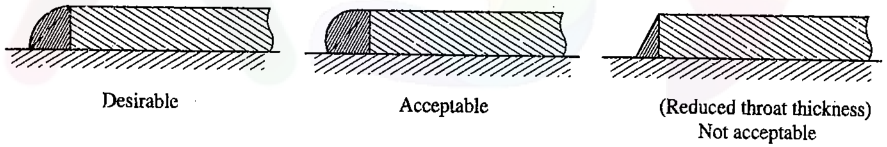

Figure 4.7

4. End fillet weld, normal to the direction of force shall be of unequal size with throat thickness not less than $0.5t$ as shown in Fig. 4.8. The difference in the thickness of weld shall be negotiated at a uniform slope.

4.5 REDUCTION IN DESIGN STRESSES FOR LONG JOINTS

If the length of the welded joint l_j is greater than $150t$, where t is throat thickness, the design capacity of weld f_{wd} shall be reduced by the factor

$$\beta_{lw} = 1.2 - \frac{0.2 l_j}{150t} \le 1.0$$

Example 4.1

A 18 mm thick plate is joined to a 16 mm plate by 200 mm long (effective) butt weld. Determine the strength of joint if

(i) a double V butt weld is used
(ii) a single V butt weld is used

Assume that Fe 410 grade plates and shop welds are used.

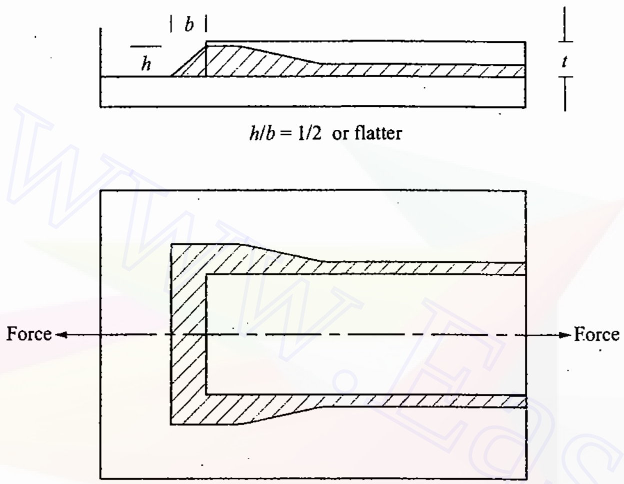

Figure 4.8

Solution:

Case (i): Double V butt weld joint

Since in such case complete penetration takes place, throat thickness = thickness of thinner plate

$t = 16$ mm.

Effective length $L_w = 200$ mm

$f_u = 410$ N/mm^2, since it is shop weld $\gamma_{mw} = 1.25$

Effective area of weld = effective length × throat thickness

$\therefore$ Design strength of weld $= \dfrac{L_w t\, f_u/\sqrt{3}}{\gamma_m}$

$$= \frac{200 \times 16 \times 410/\sqrt{3}}{1.25} = 605987 \text{ N}$$

$= 605.987$ kN **Answer**

Case (ii): Single V butt weld joint

Since penetration is not complete, effective throat thickness $t = \dfrac{5}{8} \times 16 = 10$ mm.

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$\therefore$ Design strength $= \dfrac{L_w\, t\, f_u/\sqrt{3}}{\gamma_{mw}}$

$$\frac{200 \times 10 \times 410/\sqrt{3}}{1.25} = 378742 \text{ N}$$

$= 378.742$ kN **Answer**

Example 4.2

Design a suitable longitudinal fillet weld to connect the plates as shown in Fig. 4.9 to transmit a pull equal to the full strength of small plate. Given: Plates are 12 mm thick; grade of plates Fe 410 and welding to be made in workshop.

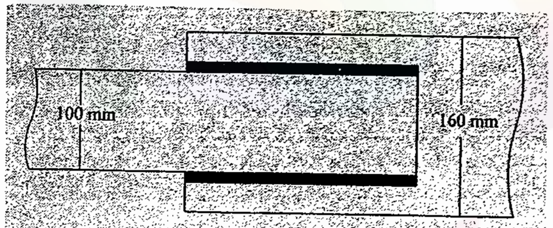

Figure 4.9

Solution:

Minimum size to be used = 5 mm

Maximum size = 12 – 1.5 = 10.5 mm

Use $s = 10$ mm fillet weld

$f_y = 410$ N/mm^2, $\gamma_{mw} = 1.25$, thickness of plate = 12 mm, breadth of plate = 100 mm

$\therefore$ Full design strength of smaller plate $= \dfrac{A_g f_y}{\gamma_{mo}}$

$f_y = 250$ MPa, $\gamma_{mo} = 1.1$

$\therefore$ Full design strength $= 12 \times 100 \times \dfrac{250}{1.1} = 272727$ N

Let effective length of welds be L_w

Assuming normal weld, throat thickness

$$t = 0.7 \times 10 = 7 \text{ mm}$$

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$\therefore$ Design strength of weld $= L_w t \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{1.25}$

$$= L_w \times 7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating it to the strength of plate, we get

$$L_w \times 7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 272727$$

$L_w = 205.7$ mm

Provide effective length of 105 mm on each side.

Example 4.3

A tie member of a roof truss consists of 2 ISA 10075, 8 mm. The angles are connected to either side of a 10 mm gusset plates and the member is subjected to a working pull of 300 kN. Design the welded connection. Assume connections are made in the workshop.

Solution:

Working Load = 300 kN

$\therefore$ Factored Load $= 300 \times 1.5 = 450$ kN

Thickness of weld:

(i) At the rounded toe of the angle section, size of weld should not exceed $= \dfrac{3}{4} \times$ thickness

$$s = \frac{3}{4} \times 8 = 6 \text{ mm}$$

(ii) At top (Ref. Fig. 4.10) the thickness should not exceed

$$s = t - 1.5 = 8 - 1.5 = 6.5 \text{ mm.}$$

Hence provide $s = 6$ mm, weld.

Each angle carries a factored pull of $\dfrac{450}{2} = 225$ kN.

Let L_w be the total length of the weld required.

Assuming normal weld, $t = 0.7 \times 6$ mm

$\therefore$ Design strength of the weld $= L_w t \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{1.25}$

$$= L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating it to the factored load, we get

$$L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 225 \times 10^3$$

$\therefore$ $L_w = 283$ mm.

Centre of gravity of the angle section is at a distance 31 mm from top.

Let L_1 be the length of top weld and L_2 be the length of lower weld. To make centre of gravity of weld to coincide with that of angle,

$$L_1 \times 31 = L_2\,(100 - 31)$$

$$\therefore \quad L_1 = \frac{69}{31} L_2$$

$$L_1 + L_2 = 283$$

i.e. $L_2\left(\dfrac{69}{31} + 1\right) = 283$

or $L_2 = 87$ mm.

$\therefore$ $L_1 = 195$ mm.

Provide 6 mm weld of $L_1 = 195$ mm and $L_2 = 87$ mm as shown in the Fig. 4.10.

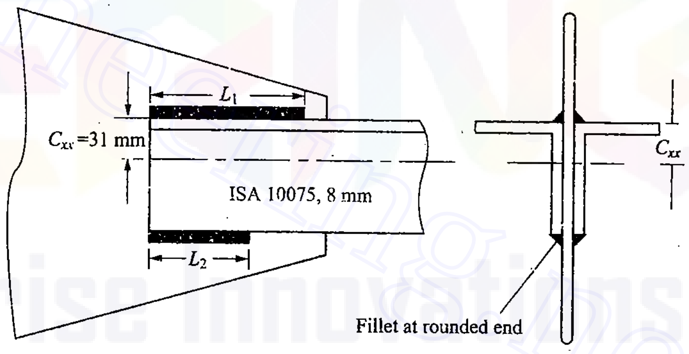

Figure 4.10

Note: In case the length available at the sides becomes insufficient, end fillet weld also may be provided. The length of end fillet should be the same on either side of centroidal axis of the angle, so that neutral axis of the weld and the section coincide (Fig. 4.11).

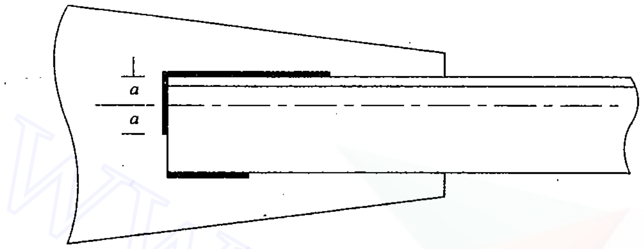

Figure 4.11

Example 4.4

Design the welded connection to connect two plates of width 200 mm and thickness 10 mm for 100 per-cent efficiency.

Solution:

Strength of plates $= \dfrac{A_g f_y}{\gamma_{mo}} = \dfrac{200 \times 10 \times 250}{1.1} = 454545$ N

Minimum size of weld = 5 mm.

Maximum size = 10 – 1.5 = 8.5 mm.

Use $s = 8$ mm weld.

Effective length of fillet welds = 2 (200 – 2 × 8)

$$L_w = 368 \text{ mm.}$$

Throat thickness $t = 0.7 \times 8$

Design strength of fillet welds $= L_w t \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{\gamma_{mw}}$

$$= 368 \times 0.7 \times 8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 390256 \text{ N}$$

∴ Slot welds are to be provided to resist a force of = 454545 – 390256 = 64289 N

Strength of the slot weld $= \dfrac{f_{wn}}{\gamma_{mw}} = \dfrac{f_u}{\sqrt{3}\gamma_{mw}}$

$$= \frac{410}{\sqrt{3} \times 1.25} = 189.37 \text{ N/mm}^2$$

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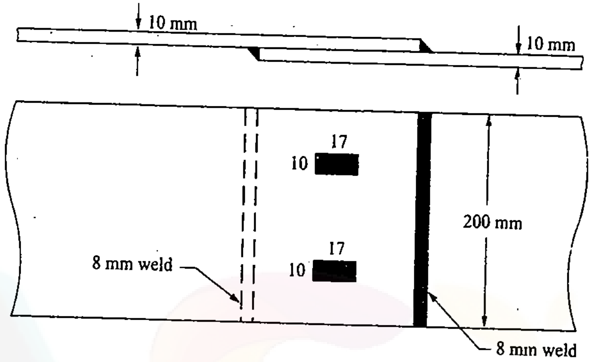

Figure 4.12

∴ Area of the slot weld required $= \dfrac{64289}{189.37} = 339.5 \text{ mm}^2$

Provide two slot welds of size 10 mm × 17 mm as shown in Fig. 4.12.

Example 4.5

A tie member consists of two ISMC 250. The channels are connected on either side of a 12 mm thick gusset plate. Design the welded joint to develop the full strength of the tie. However the overlap is to be limited to 400 mm.

Solution:

For ISMC 250, [From steel tables]

Thickness of web = 7.1 mm

Thickness of flange = 14.1 mm

Sectional area = 3867 mm^2

Tensile design strength of each channel $= \dfrac{A_g f_y}{1.1} = \dfrac{3867 \times 250}{1.1} = 878864$ N

Weld thickness:

Minimum thickness = 3 mm.

Maximum thickness = 7.1 – 1.5 = 5.6 mm

Provide $s = 4$ mm weld.

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$\therefore$ Throat thickness, $t = 0.7 \times 4 = 2.8$ mm

Strength of weld $= L_w t \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{\gamma_{mw}}$

$$= L_w \times 2.8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating strength of weld to tensile strength of the channel, we get

$$L_w \times 2.8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 878804$$

$\therefore$ $L_w = 1658$ mm.

Since allowable length is limited to 400 + 400 mm it needs slot weld. The arrangement can be as shown in the figure with two slots of length 'x'. Then

$$400 + 400 + (250 - 2 \times 30) + 4x = 1658$$

$$\therefore \quad x = 167 \text{ mm.}$$

(as $2s$ length of weld will be in effective at each term)
Provide $x = 185$ mm as shown in Fig. 4.13.

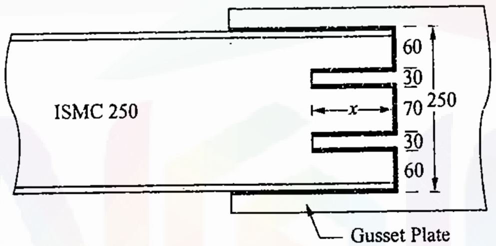

Figure 4.13

4.6 ECCENTRIC CONNECTION – PLANE OF MOMENT AND THE PLANE OF WELDS IS THE SAME

Figure 4.14(a) shows a typical case. The eccentric load P is equivalent to

(i) a direct load P at the centre of gravity of the group of weld and
(ii) a twisting moment $P \times e$.

Let a weld of uniform size be provided throughout and 't' be the effective throat thickness. If 'd' is the depth of weld and 'b' is the width as shown in the figure, the direct shear stress in the weld is

$$q_1 = \frac{P}{(2b + d)t}$$

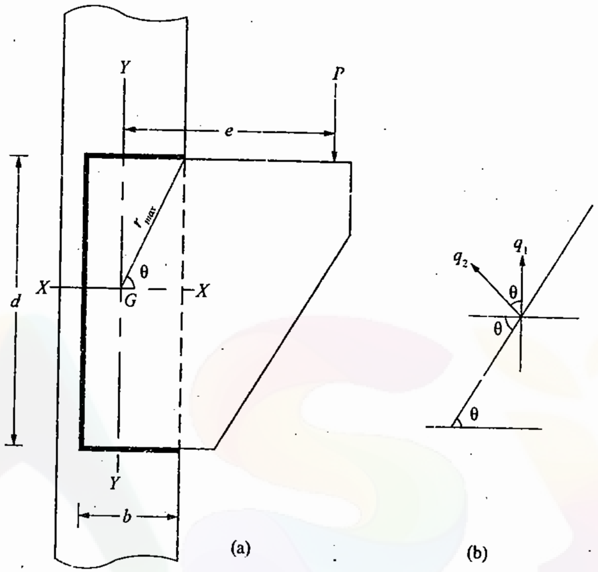

Figure 4.14

The stress in the weld due to twisting moment is the maximum in the weld at the extreme distance from the centre of gravity of the group of weld and acts in the direction perpendicular to the radius vector. The maximum stress due to the moment

$$q_2 = \frac{P \times e \times r_{max}}{I_{zz}}$$

where r_{max} is the distance of the extreme weld from the c.g. of the group.

$I_{zz} = I_{xx} + I_{yy}$, the polar moment of inertia. The vector sum of the stress is $q = \sqrt{q_1^2 + q_2^2 + 2q_1 q_2 \cos\theta}$

For the safe design it should be less than the resistance per unit area.

The above expression is equal to [Ref. Fig. 4.14(b)]

$$q_x = q_2 \sin\theta$$

$$q_y = q_1 + q_2 \cos\theta$$

$$\therefore \quad q = \sqrt{q_x^2 + q_y^2}$$

Example 4.6

Determine the maximum load that can be resisted by the bracket shown in Fig. 4.15, by fillet weld of size 6 mm, if it is shop welding.

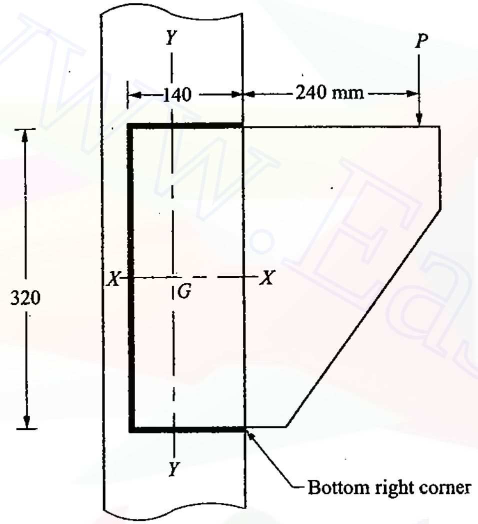

Figure 4.15

Solution:

Size of weld = 6 mm.

$\therefore$ Throat thickness = 0.7 × 6 = 4.2 mm

Consider the area of the weld which has channel shape and has width 4.2 mm throughout.

Total area of weld = 320 × 4.2 + 140 × 4.2 × 2

$= 600 \times 4.2 \text{ mm}^2$

Due to symmetry, centroidal *x-x* axis is at the mid height of vertical weld. Let centroidal *y-y* axis be at a distance $\bar{x}$ from the vertical weld. Then

$$\bar{x} = \frac{140 \times 4.2 \times 70 \times 2}{600 \times 4.2} = 32.67 \text{ mm}$$

$$I_{xx} = \frac{1}{12} \times 4.2 \times 320^3 + 140 \times 4.2 \times 160^2 \times 2$$

$$= 41574400 \text{ mm}^4$$

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$$I_{yy} = 320 \times 4.2 (32.67)^2 + 2\left[\frac{1}{12} \times 140^3 \times 4.2 + 4.2 \times 140 (70 - 32.67)^2\right]$$

$$= 4994080 \text{ mm}^4$$

$$\therefore \quad I_{zz} = I_{xx} + I_{yy} = 46568480 \text{ mm}^4$$

$$r_{max} = \sqrt{160^2 + (140 - 32.67)^2} = 192.66 \text{ mm.}$$

$$\tan\theta = \frac{160}{140 - 32.67} \qquad \therefore \quad \theta = 56.15°$$

Eccentricity of load from the centre of gravity of weld

$$e = 240 + 140 - 32.67 = 347.33 \text{ mm.}$$

Let *P* be in kilo newtons.

Direct shear stress $q_1 = \dfrac{P \times 1000}{\text{Total area}} = \dfrac{P \times 1000}{600 \times 4.2}$

$= 0.3968\,P \text{ N/mm}^2$

Shear stress at extreme edge due to torsional moment

$$q_2 = \frac{P \times e \times r_{max}}{I_{zz}}$$

$$= \frac{P \times 1000 \times 347.33 \times 192.66}{46568480}$$

$$= 1.4370\,P \text{ N/mm}^2$$

$\therefore$ Resultant stress $= \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$

$= P\sqrt{0.3968^2 + 1.4370^2 + 2 \times 0.3968 \times 1.4370 \times \cos 56.15}$

$= 1.69046\,P$

Weld can resist a stress of $= \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{1.25} = \dfrac{410}{\sqrt{3} \times 1.25} = 189.37 \text{ N/mm}^2$

Equating maximum stress to resisting stress, we get,

$1.69046\,P = 189.37$

$\therefore$ $P = 112$ kN **Answer**

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Example 4.7

In the example 4.6, if the load is 100 kN inclined at 30° to vertical in clockwise direction, check whether the weld is safe.

Solution:

The horizontal component of load $= 100 \sin 30°$

$= 50$ kN

The vertical component $= 100 \cos 30° = 86.6$ kN

Twisting moment on the joint $= -50 \times 160 + 86.6 \times 347.33$

$= 22078.77$ kN-mm

$= 22078.77 \times 10^3$ N-mm

Direct shear stress in horizontal direction $= \dfrac{50 \times 1000}{2520}$

$= 19.841$ N/mm^2

Direct shear stress in vertical direction $= \dfrac{86.6 \times 1000}{2520}$

$= 34.365$ N/mm^2

Maximum shear stress due to twisting moment $= \dfrac{22078.77 \times 10^3}{46568480} \times 192.66$

$= 913.343$ N/mm^2

The maximum resultant shear act at bottom right corner as shown in Fig. 4.16.

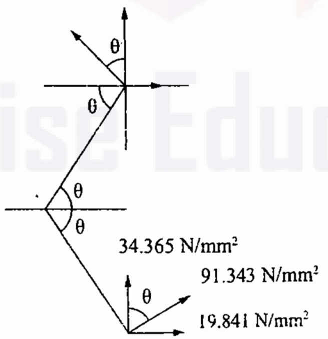


Figure 4.16

In this case $\theta = 56.15°$

$$q_h = 19.841 + 91.343 \sin 56.15° = 95.701 \text{ N/mm}^2$$

$$q_v = 34.365 + 91.343 \cos 56.15 = 85.245 \text{ N/mm}^2$$

$$\therefore \quad q = \sqrt{95.701^2 + 85.245^2} = 128.162 \text{ N/mm}^2$$

Resistance capacity $= \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{1.25} = \dfrac{410}{\sqrt{3}} \times \dfrac{1}{1.25}$

$= 189.37$ N/mm$^2 > 128.164$ N/mm^2

Hence the weld is safe.

Example 4.8

The 10 mm thick bracket plate shown in Fig. 4.17 is connected with the flange of column ISHB 300 @ 577 N/m. Find the size of the weld to transmit a factored load of 250 kN.

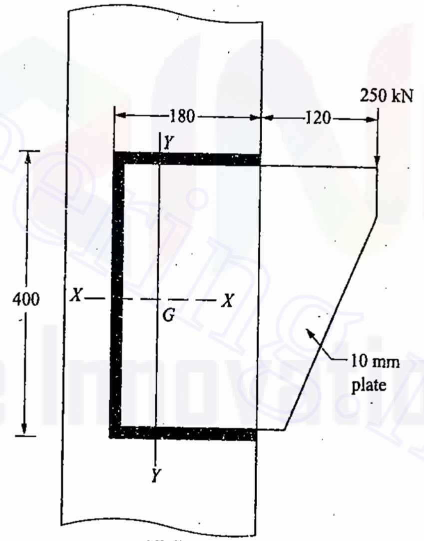


Figure 4.17

Solution:

Let 't' be the throat thickness of the weld required and $\bar{x}$ be the distance of c.g. of weld from vertical weld. Then

$$\text{Area of the weld} = 400\,t + 180\,t \times 2 = 760\,t$$

$$\bar{x} = \frac{2\times180t\times90}{760t} = 42\ 63\text{ mm.}$$

$$I_{xx} = \frac{1}{12}\times400^3\times t + 180t\times200^2\times2 = 19733333t\text{ mm}^4$$

$$I_{yy} = 400t\times42.63^2 + 2\left[\frac{1}{12}\times t\times180^3 + 180t\times(90-42.63)^2\right]$$

$$= 2506737t\text{ mm}^4$$

$$\therefore\ I_{zz} = 22240070t\text{ mm}^4$$

Distance of extreme point of the weld from c.g.

$$r_{\max} = \sqrt{200^2 + (180-42.63)^2} = 242.63\text{ mm}$$

$$\tan\theta = \frac{200}{180-42.63} = 1.4559$$

$$\theta = 55.517°$$

Eccentricity $e = 120 + 180 - 42.63 = 257.37$ mm.

$$\therefore\ \text{Direct shear stress} = q_1 = \frac{250\times10^3}{760t} = \frac{328.95}{t}\text{ N/mm}^2$$

Maximum shear stress due to twisting moment

$$q_2 = \frac{P\times e\times r_{\max}}{I_{zz}} = \frac{250\times10^3\times257.37\times242.63}{22240070t}$$

$$= \frac{701.950}{t}\text{ N/mm}^2$$

$$\therefore q = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$$

$$= \sqrt{\left(\frac{328.95}{t}\right)^2 + \left(\frac{701.95}{t}\right)^2 + 2\frac{328.95}{t}\times\frac{701.950}{t}\cos 55.517}$$

$$= \frac{928.656}{t}$$

$$\text{Resistance of the weld} = \frac{f_u}{\sqrt{3}}\times\frac{1}{1.25} = \frac{410}{\sqrt{3}}\times\frac{1}{1.25}$$

$$= 189.37\text{ N/mm}^2$$

Equating maximum shear to it, we get $\frac{928.656}{t} = 189.37$

$$\therefore\ t = 4.904\text{ mm}$$

$$\therefore\ \text{Size of normal fillet} = \frac{4.904}{0.7} = 7.005\text{ mm}$$

$\therefore$ Provide 8 mm fillet weld. **Answer**

4.7 COMBINED AXIAL AND SHEAR STRESS

If a weld is subjected to axial stresses, compression or tension due to axial force or bending moment simultaneously with shear, IS 800-2007 has made the following provisions:

Fillet weld: The equivalent stress f_e shall satisfy the following:

$$f_e = \sqrt{f_a^2 + 3q^2} \le \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

where f_a – axial stress, direct or due to bending.

and q – shear stress due to shear force or tension.

Check for combination of stresses need not be done for:

(a) side fillet welds joining cover plates and flange plates, and
(b) fillet welds where sum of normal and shear stresses does not exceed $f_{wd} = \frac{f_u}{\sqrt{3}}\times\frac{1}{\gamma_{mw}}$.

Butt Welds

Check for the combination of stresses in butt weld need not be carried out provided that

(a) butt welds are axially loaded.
(b) in single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 per cent of the design shear stress.

4.8 ECCENTRIC CONNECTION–MOMENT AT RIGHT ANGLES TO THE PLANE OF WELD

Figure 4.18 shows the typical case in which P is the factored load at an eccentricity 'e'. Let h be the effective depth of fillet weld. Fillet welding is on both sides of bracket plate. Hence if throat thickness is 't', effective area of the weld = $2ht$.

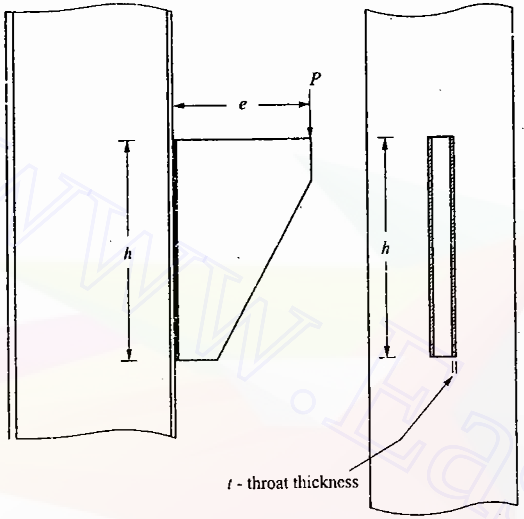


Figure 4.18

Then,

direct shear stress $q = \frac{P}{2ht}$

bending stress at the extreme edge of the weld

$$f = \frac{M}{Z} = \frac{P \cdot e}{\frac{1}{6}\left(2th^2\right)} = \frac{6Pe}{2th^2}$$

$\therefore$ Equivalent stress

$$f_e = \sqrt{f^2 + 3q^2} \leq \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

For the purpose of finding the effective depth h required, first depth required for bending only may be found. To take care of shear also, increase this value by about 10 percent i.e.,

$$h' = \sqrt{\frac{6M}{2t\, f_{wd}}}$$

and hence try, $h = 1.1\,h'$.

Example 4.9

Design a suitable fillet weld for the bracket shown in Fig. 4.18, if working load P = 100 kN and eccentricity e = 150 mm. Thickness of the bracket plate is 12 mm and the column used is ISHB 300 @ 618 N/m.

Solution:

Load = 100 kN

$\therefore$ Factored load, $P = 100 \times 1.5 = 150$ kN

Thickness of flange of ISHB 300 @ 618 N/m is 10.6 mm.

$\therefore$ Minimum size of weld = 5 mm.

Use 8 mm weld on each side of bracket plate.

Throat thickness, $t = 0.7 \times 8$ mm.

Resistance of weld $f_{wd} = \frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}} = \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 189.37$ N/mm^2.

$\therefore$ Depth of weld required to resist bending alone $= h' = \sqrt{\frac{6 \times 150 \times 10^3 \times 150}{2 \times 0.7 \times 8 \times 189.37}} = 252.3$ mm

About 10 per cent extra depth is to be provided.

Let h = 280 mm.

Check for the stresses:

Direct shear stress $q = \frac{P}{2 \times t \times h} = \frac{150 \times 10^3}{2 \times 0.7 \times 8 \times 280} = 47.83$ N/mm^2

Bending stress $f = \frac{M}{Z} = \frac{6M}{2t \times h^2} = \frac{6 \times 150 \times 10^3 \times 150}{2 \times 0.7 \times 8 \times 280^2}$

$= 153.744$ N/mm^2

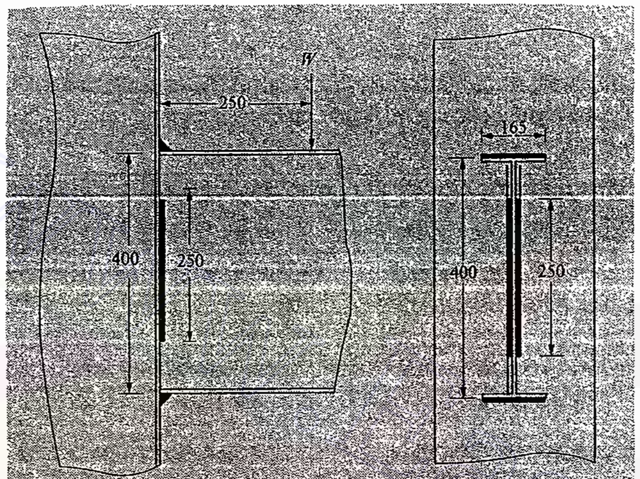
$$\therefore \sqrt{f^2 + 3q^2} = \sqrt{153.744^2 + 3 \times 47.83^2}$$

$$= 174.6 \text{ N/mm}^2 < 189.37 \text{ N/mm}^2$$

Hence design is safe.

Example 4.10

An I section bracket is connected to a column by welds as shown in Fig. 4.19. Determine the load which can be safely carried. The size of the web weld is 5 mm while the size of flange weld is 10 mm. Assume field welds.



All dimensions in mm

Figure 4.19

Solution:

Assuming normal fillet welds,

throat thickness of flange welds = $0.7 \times 10 = 7$ mm

throat thickness of web welds = $0.7 \times 5 = 3.5$ mm

$\therefore$ Total throat area of weld $= 165 \times 7 \times 2 + 250 \times 3.5 \times 2 = 4060$ mm^2

$$I_{xx} = 165 \times 7 \times 200^2 \times 2 + \frac{1}{12} \times 3.5 \times 250^3 \times 2 = 101514583 \text{ mm}^4$$

Bending moment to be transferred is $= P \times 250$ kN-mm, if P is factored load in kN

$= P \times 250 \times 1000$ N-mm.

Consider flange weld which is subjected to maximum stress

$$q_v = \frac{P \times 1000}{4060} \text{ N/mm}^2$$

$$= 0.2463P \text{ N/mm}^2$$

Due to bending,

$$q_h = \frac{250 \times 1000P}{101514583} \times 200$$

$$= 0.49254\, P \text{ N/mm}^2$$

$$\therefore \quad q = \sqrt{q_v^2 + q_h^2} = P\sqrt{0.2463^2 + 0.49254^2} = 0.5507\, P$$

Equating it to design stress $\frac{f_u}{\sqrt{3}} \times \frac{1}{1.5}$, we get $0.5507 = \frac{410}{\sqrt{3}} \times \frac{1}{1.5}$

$$\therefore \quad P = 286.57 \text{ kN}$$

[*Note:* γ_w for field welds = 1.5]

$\therefore$ Working load that can be permitted is, $W = \frac{P}{1.5} = \frac{286.57}{1.5} = 191.04$ kN **Answer**

* * *

Questions

1. What are the advantages and disadvantages of welded connections?
2. Neatly sketch the following welded connections:
   (a) Butt weld (groove weld) – single *V*, double *V*
   (b) Fillet weld
   (c) Slot weld
   (d) Plug weld.
3. Two 12 mm thick plates are joined by 160 mm long (effective) butt weld. Determine the strength of joint if
   (a) Single U butt weld is used.
   (b) Double U butt weld is used.
4. Design a suitable longitudinal fillet weld to connect 120×8 mm plate to 150×10 mm plate to transmit a pull equal to the full strength of small plate. Assume welding is to be made in the field.
5. A tie member of a roof truss consists of 2 ISA 9060, 10 mm. The angles are connected on the either side of 12 mm gusset plate and the member is subjected to a factored pull of 350 kN. Design the welded connection. Assume welding is to be made in the workshop.
6. Two plates 180 mm wide and 8 mm thick are to be connected by welding, using shop welds, design the connection.

7. A bracket plate is welded to the flange of a column section ISHB 300 @ 577 N/m as shown in Fig. 4.14. If width of weld is 200 mm, depth 260 mm and eccentricity from the face of column is 80 mm, determine the size of the weld to support a factored load of 165 kN.
8. Design a suitable fillet weld for the bracket shown in Fig. 4.18, if it has to transfer a factored load of 200 kN at an eccentricity of 160 mm. Use shop fillet welds.
9. Design a column bracket shown in Fig. 4.20 to take a load of 160 kN at an eccentricity of 200 mm. The size of the web welds should be half the size of the flange welds.

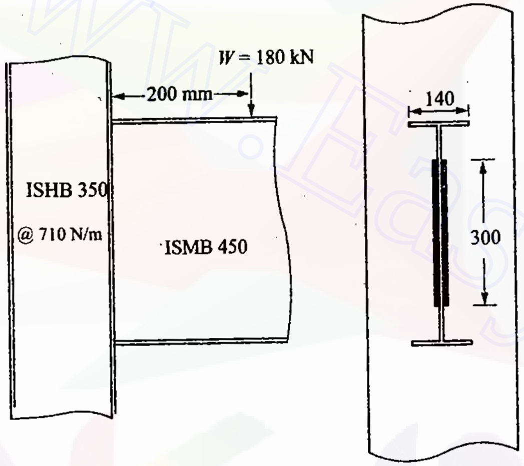


Figure 4.20

5

DESIGN OF TENSION MEMBERS

Tension members are also known as tie members. The form of a tension member is governed to a large extent by the type of the structure of which it is a part and by the method of joining it to the adjacent member of the structure. In general the section should be compact and in order to minimize stress concentration it should be so arranged that as large portion of it as possible is connected to the gusset plates. The common shapes used are shown in Fig. 5.1.

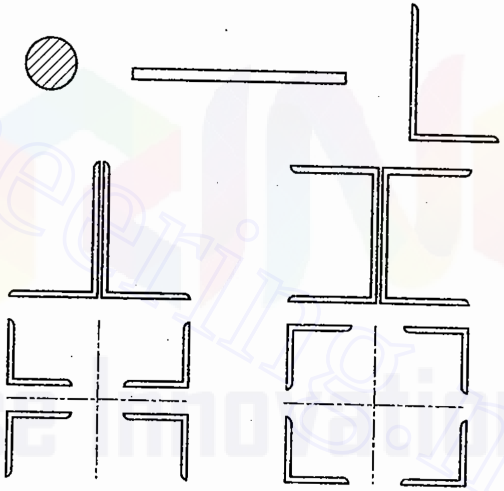


Figure 5.1 Shapes of tension members.

5.1 DESIGN STRENGTH OF A TENSION MEMBER

The design strength of a tension member is the lowest of the following:

(a) Design strength due to yielding of gross section T_{dg}.
(b) Rupture strength of critical section, T_{dn} and
(c) The block shear T_{db}.

5.1.1 Design Strength Due to Yielding of Gross Section

This strength is given by

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

where, f_y = yield stress of the material
A_g = gross area of the cross-section
γ_{mo} = partial safety factor for failure in tension by yielding = 1.1.

5.1.2 Design Strength Due to Rupture of Critical Section

As explained in chapter 3, this strength for plates is

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$$

where A_n net effective area at critical section

$$= \left[b - nd_o + \sum \frac{p_{si}^2}{4g_i}\right] t$$

For threaded rods and bolts

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$$

where A_n = net area at the threaded section.

$$= \frac{\pi}{4}(d - 0.9382p)^2, \text{ where } p \text{ is pitch of thread}$$

$$\approx 0.78 \frac{\pi}{4} d^2 \text{ for ISO threads}$$

Single Angle [Ref. Fig. 5.2]

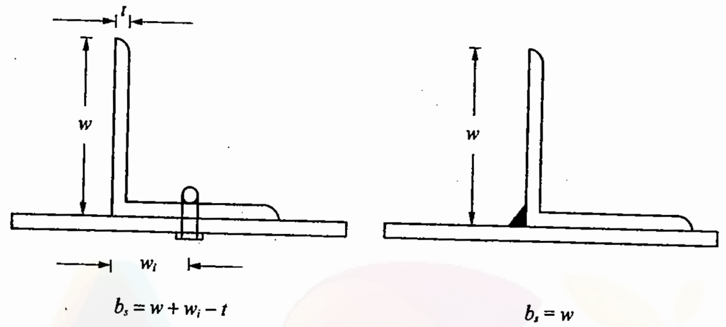


Figure 5.2

As the effectiveness of outstanding leg is less, the design strength as governed by rupture at net section is given by

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

where, A_{nc} = net area of the connected leg
A_{go} = gross area of the outstanding leg

and

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$$

where w = outstanding leg width.
b_s = shear leg width, as shown in Fig. 5.2.
and L_c = length of the end connection, that is, the distance between outermost bolt in the end joint measured along the load direction or length of the weld along the load direction.
t = thickness of leg.

For preliminary design IS code recommends the following formula:

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{ml}}$$

where α = 0.6 for one or two bolts
= 0.7 for three bolts
= 0.8 for four or more bolts along the length of connection or equivalent weld length.

However, if it is difficult to find equivalent weld length, designers have to judge this.

Other Sections

The rupture strength, T_{dn} of the double angles, channels, I-sections etc., may be calculated by the same equation as for single angle, but with b_s taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg.

5.1.3 Design Strength Due to Block Shear

At the connected end, failure of a tension member may occur along a path involving shear along one plane and tension on a perpendicular plane along the fastener. This type of failure is known as block failure.

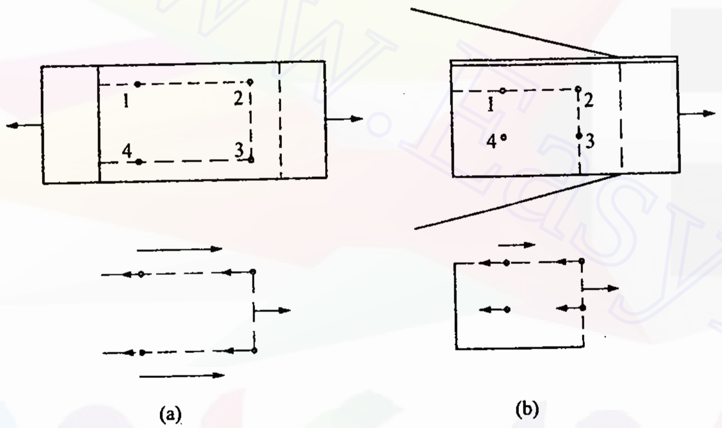


Figure 5.3

Referring to Fig. 5.3(a), shear failure occurs along 1-2 and 3-4 whereas tension failure occurs along 2-3.

Referring to Fig. 5.3(b), shear failure occurs along 1-2 and tension failure along 2-3.

IS 800-2007, recommends the following block shear strength T_{db} if bolted connections are used. It shall be smaller of

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

or

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

where, A_{vg} and A_{vn} = Minimum gross and net area in shear (1-2, 3-4 in Fig. 5.3(a), 1-2 in Fig. 5.3(b))

A_{tg} and A_{tn} = Minimum gross and net area in tension [2-3 as shown in Fig. 5.3].

Note: The block shear strength, T_{db} shall be checked for welded end connections by taking an appropriate section around the end weld.

Example 5.1

Determine the design tensile strength of the plate 130 mm × 12 mm with the holes for 16 mm diameter bolts as shown in Fig. 5.4. Steel used is of Fe 410 grade quality.

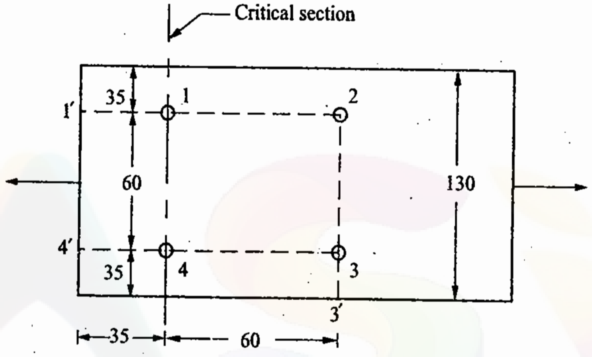


Figure 5.4

Solution:

Strength of the plate is the least of

(a) Yielding of gross section
(b) Rupture of critical section
(c) The block shear strength

(a) From consideration of yielding:

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

Now, $A_g = 130 \times 12 = 1560\ \text{mm}^2$, $f_y = 250\ \text{N/mm}^2$, $\gamma_{mo} = 1.1$

$$\therefore \quad T_{dg} = \frac{1560 \times 250}{1.1} = 354545\ \text{N} = 354.545\ \text{kN}$$

(b) From the consideration of rupture along the critical section:

Critical section is having two holes.

Diameter of holes = 16 + 2 = 18 mm.

$\therefore \quad A_n = (130 - 2 \times 18) \times 12 = 1128\ \text{mm}^2$

Strength of member from the consideration of rupture

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} = \frac{0.9 \times 1128 \times 410}{1.25}$$

$$= 332986 \text{ N} = 332.986 \text{ kN}$$

(c) Block shear strength:

$A_{vg} = 2 \times (35 + 60) \times 12 = 2280 \text{ mm}^2$ $\quad A_{tg} = 60 \times 12 = 720 \text{ mm}^2$

$A_{vn} = (35 + 60 - 1.5 \times 18) \times 12 \times 2 = 1632 \text{ mm}^2$ $\quad A_{tn} = (60 - 18) \times 12 = 504 \text{ mm}^2$

The block shear strength is the least of the following two:

(1)
$$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}\right]$$

$$= \frac{1140 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 816 \times 410}{1.25}$$

$$= 350570 \text{ N} = 350.570 \text{ kN.}$$

(2)
$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= 350570 \text{ N} = 350.570 \text{ kN}$$

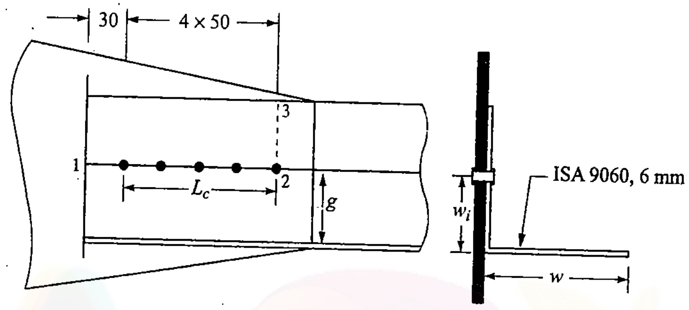
$$\therefore \quad T_{db} = 441.784 \text{ kN}$$

Strength of plate = 332.986 kN

Example 5.2

A single unequal angle ISA 9060, 6 mm is connected to a 10 mm gusset plates at the ends with 5 nos. of 16 mm bolts to transfer tension [Ref. Fig. 5.5]. Determine the design tensile strength of the angle

(a) if the gusset is connected to 90 mm leg.
(b) if the gusset is connected to 60 mm leg.



Given: g = 50 mm, if 90 mm leg is connected, $b_s = w + w_1 - t$
= 30 mm, if 60 mm leg is connected

Figure 5.5

Solution:

(a) 90 mm leg is connected to gusset:

(i) Strength as governed by yielding of gross section:

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

$Ag = 865 \text{ mm}^2$ (from table)

$$\therefore \quad T_{dg} = \frac{865 \times 250}{1.1} = 196364 \text{ N} = 196.364 \text{ kN.}$$

(ii) Strength as governed by tearing at critical section:

Net area of connected leg $= A_{nc} = \left(90 - \frac{6}{2}\right) \times 6 = 522 \text{ mm}^2$

Gross area of outstanding leg $= A_{go} = \left(60 - \frac{6}{2}\right) \times 6 = 342 \text{ mm}^2$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

w = length of outstanding leg = 60 mm
w_i = 50 mm
b_s = 60 + 50 − 6 = 104 mm
L_c = 4 × 50 = 200 mm

$$\therefore \quad \beta = 1.4 - 0.076 \times \frac{60}{6} \times \frac{250}{410} \times \frac{104}{200}$$

$$= 1.159 \leq \frac{f_u}{f_y} \times \frac{\gamma_{mo}}{\gamma_{ml}} \geq 0.7$$

Hence $\beta = 1.159$

$$\therefore \quad T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.159 \times 342 \times 250}{1.1}$$

$$= 244180 \text{ N} = 244.180 \text{ kN.}$$

(iii) Block shear strength

Failure may take along section 1-2-3.

$d_0 = 16 + 2 = 18$ mm, tearing length in tension $= 90 - 50 = 40$ mm

$A_{vg} = 230 \times 6 = 1380 \text{ mm}^2$ $\quad A_{vn} = (230 - 4.5 \times 18) \times 6 = 894 \text{ mm}^2$

$A_{tg} = 40 \times 6 = 240 \text{ mm}^2$ $\quad A_{tn} = (40 - 0.5 \times 18) \times 6 = 186 \text{ mm}^2$

$\therefore$ Block shear strength is smaller of the following two values:

(a)
$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25}$$

$$= 235985 \text{ N} = 235.985 \text{ kN}$$

or

(b)
$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.1}$$

$$= 206913 \text{ N} = 206.913 \text{ kN}$$

$$\therefore \quad T_{db} = 206.913 \text{ kN}$$

Thus, when 90 mm leg is connected to gusset, the strength of the plate is the least of 196.364 kN, 244.180 kN and 206.913 kN, i.e.,

Strength of the plate is 196.364 kN **Answer**

(b) When 60 mm leg is connected to gusset plate:

(i) Strength as governed by yielding of gross area

$T_{dg} = 196.364$ kN as in case (a)

(ii) Strength as governed by tearing at critical section:

Net area of connected leg $= \quad A_{nc} = \left(60 - \frac{6}{2}\right) \times 6 = 342 \text{ mm}^2$

Gross area of outstanding leg $= A_{go} = \left(90 - \frac{6}{2}\right) \times 6 = 522 \text{ mm}^2$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$w = 90$ mm $\quad w_i = 30$ mm

$b_s = 90 + 30 - 6 = 114$ mm

$L_c = 50 \times 4 = 200$ mm

$$\beta = 1.4 - 0.076 \times \frac{90}{6} \times \frac{250}{410} \times \frac{114}{200}$$

$$1.004 < \frac{f_u}{f_y} \frac{\gamma_{mo}}{\gamma_{ml}} > 0.7$$

$$\therefore \quad \beta = 1.004$$

$$\therefore \quad T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 342 \times 410}{1.25} + \frac{1.004 \times 522 \times 250}{1.1}$$

$$= 220069 \text{ N} = 220.069 \text{ kN}$$

(iii) Block shear strength:

Tearing length in tension $= 60 - 30 = 30$ mm

$A_{vg} = 230 \times 6 = 1380 \text{ mm}^2$ $\quad A_{vn} = (230 - 4.5 \times 18) \times 6 = 894 \text{ mm}^2$

$A_{tg} = 30 \times 6 = 180 \text{ mm}^2$ $\quad A_{tn} = (30 - 18) \times 6 = 72 \text{ mm}^2$

Block shear strength is the smaller of the following two values:

(a)
$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 72 \times 410}{1.25}$$

$$= 202332 \text{ N} = 202.332 \text{ kN}$$

(b)
$$T_{db} = \frac{0.9 A_{vx} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.1}$$

$$= 193277 \text{ N} = 193.277 \text{ kN} < 202.332 \text{ kN}$$

$$\therefore \quad T_{db} = 193.277$$

Strength of member is the least of 196.364 kN, 220.069 kN and 193.277 kN. i.e.,

Strength of member = 193.277 kN **Answer**

Example 5.3

Instead of single angle, if two angles are used in the connection as described in example 5.2, determine the design tensile strength

(a) if two such angles are connected to the same side of the gusset plate through the 90 mm leg.
(b) if two such angles are connected to the opposite sides of the gusset through 60 mm leg.

Solution:

These two connections are shown in Fig. 5.6.

It may be noted that according to IS 800-2007 formulae, the double angle strength would be twice that of single angle as obtained in example 5.2. Earlier code used to give strength more than double. If tacking bolts are provided or not there is no change in strength of tensile member. These provisions have been confirmed by test results and finite element results. These are some of the major changes suggested in IS 800-2007.

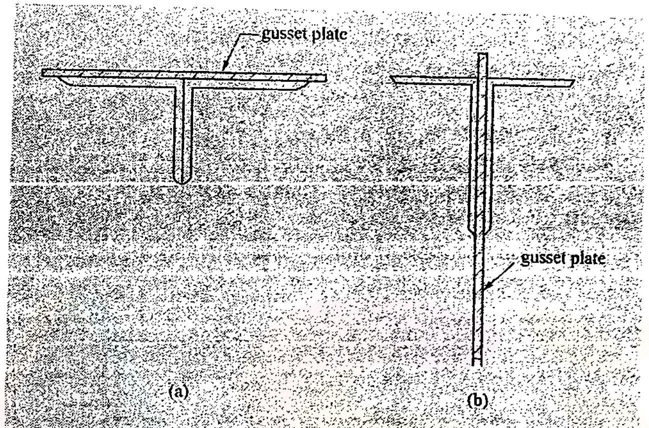


Figure 5.6

Example 5.4.

Determine the design tensile strength of 160 × 8 mm plate with the holes for 16 mm bolts as shown in Fig. 5.7. Plates are of steel, grade Fe 415.

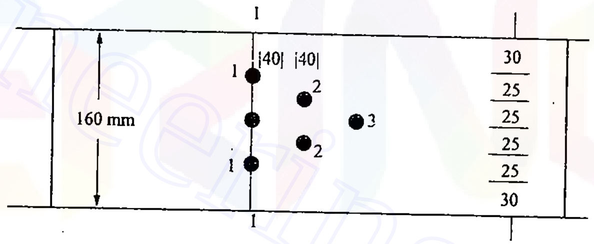


Figure 5.7

Solution:

(a) Strength from the consideration of yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{160 \times 8 \times 250}{1.1} = 290909 \text{ N} = 290.909 \text{ kN}$$

(b) Strength from the consideration of rupture along the critical section:

$$A_n = \left[b - nd_0 + \sum \frac{p_{si}^2}{4g_i}\right] t$$

$b = 160$ mm, $d_0 = 16 + 2 = 18$ mm, $p_{si} = 40$ mm, $g_i = 25$ mm

(i) Along section 1-1-1-1:

$$A_n = (160 - 3 \times 18)\, 8 = 848 \text{ mm}^2$$

(ii) Along section 1-1-2-2-1-1:

$$A_n = \left[160 - 4 \times 18 + \frac{2 \times 40^2}{4 \times 25}\right] \times 8 = 960 \text{ mm}^2$$

(iii) Along section 1-1-2-3.2-1-1:

$$A_n = \left[160 - 5 \times 18 + \frac{4 \times 40^2}{4 \times 25}\right] \times 8 = 1072 \text{ mm}^2$$

A_n to be selected is 848 mm^2

$$\therefore \quad T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} = \frac{0.9 \times 848 \times 410}{1.25} = 250330 \text{ N}$$

$$= 250.330 \text{ kN.}$$

$\therefore$ Strength of plate = 250.330 kN. **Answer**

Example 5.5

Determine the tensile strength of a roof truss member 2 ISA 9060, 6 mm connected to the gusset plate of 8 mm thickness by 4 mm weld as shown in Fig. 5.8. The effective length of weld is 200 mm.

Solution:

Gross area of angles, $A_g = 2 \times 865 = 1730$ mm^2

Area of the connected leg, $A_{nc} = 2\left(90 - \frac{6}{2}\right) \times 6 = 1044$ mm^2

Area of the outstanding leg, $A_{go} = 2\left(60 - \frac{6}{2}\right) \times 6 = 684$ mm^2

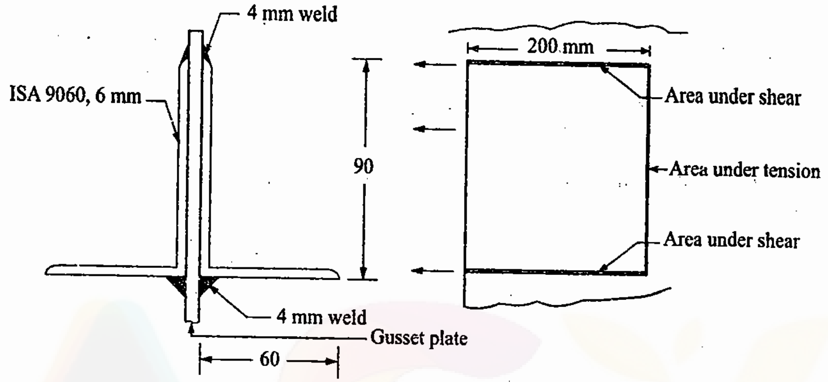


Figure 5.8

(i) Strength governed by yielding

$$= \frac{A_g f_y}{\gamma_{mo}} = \frac{1730 \times 250}{1.1} = 393182 \text{ N} = 393.182 \text{ kN}$$

(ii) Strength of the plate in rupture at critical section:

$$T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

Now, $$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_w}$$

In this case $w = 90$ mm, $t = 6$ mm, $f_y = 250$ MPa, $f_u = 410$ MPa, $b_s = w = 90$ mm, $L_w = 200$ mm.

$$\therefore \quad \beta = 1.4 - 0.076 \times \frac{90}{6} \times \frac{250}{410} \times \frac{90}{200}$$

$$= 1.087 \text{ which is} < \left(\frac{f_u\, \gamma_{mo}}{f_y\, \gamma_{ml}}\right) > 0.7$$

$\therefore$ $\beta = 1.087$, since $f_u = 410$ N/mm^2, $f_y = 250$ N/mm^2, $\gamma_{mo} = 1.1$ and $\gamma_{ml} = 1.25$

Hence $$T_{dn} = \frac{0.9 \times 410 \times 1044}{1.25} + \frac{1.087 \times 684 \times 250}{1.1}$$

$$= 477168 \text{ N} = 477.168 \text{ kN}$$

(iii) Strength governed by block shear:

One shear leg and one tension face failure can occur. There are two angles.

Area under shear, $A_{vg} = A_{vn} = 2 \times 200 \times (4 + 4) = 3200 \text{ mm}^2$

Area under tension at failure $= A_{tg} = A_{tn} = 90 \times 2 \times 4 = 720 \text{ mm}^2$.

$$= \frac{3200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 720 \times 410}{1.25}$$

$$= 632435 \text{ N} = 632.435 \text{ kN}$$

or

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \times 1.25} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 3200 \times 410}{\sqrt{3} \times 1.25} + \frac{720 \times 250}{1.1}$$

$$= 709025 \text{ N} = 709.025 \text{ kN}$$

$\therefore$ Block shear strength $T_{db} = 632.435$ kN

$\therefore$ Strength of tension member = 392.727 kN **Answer**

5.2 DESIGN PROCEDURE

The following design procedure may be adopted.

1. Find the required gross area to carry the factored load considering the strength in yielding. i.e.,

$$A_g = \frac{T_u}{(f_y / \gamma_{mo})} = \frac{1.1 T_u}{f_y}$$

where T_u = factored tensile force.

2. Select suitable shape of the section depending upon the type of structure and the location of the member such that gross area is 25 to 40 per cent more than A_g calculated.
3. Determine the number of bolts or the welding required and arrange.
4. Find the strength considering:
   (a) Strength in yielding of gross area
   (b) Strength in rupture of critical section and
   (c) Strength in block shear.

Usually, if minimum edge distance and minimum pitch are maintained, strength in yielding is the least value, hence the design is safe if A_g provided > A_g required.

5. The strength obtained should be more than factored tension. If it is too much on higher side or the strength is less than factored tension, the section may be suitably changed and checked.
6. IS 800-2007 also recommends the check for slenderness ratio of tension members as per the Table 5.1.

Table 5.1 Maximum values of effective slenderness ratios
[From Table 3 of IS 800-2007]

Sl. No.	Member	Max. l/r
1	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
2	A member normally acting as a tie in a roof truss or a bracing system not considered effective when subject to possible reversal of stress into compression resulting from the action of wind or earthquake forces	350
3	Members always under tension other than pretensioned members	400
4	Tension members, such as bracings, pretensioned to avoid sag, need not satisfy the maximum slenderness ratio limit	No limit

Example 5.6

Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225 kN. The member is subjected to the possible reversal of stress due to the action of wind. The effective length of the member is 3 m. Use 20 mm shop bolts of grade 4.6 for the connection.

Solution:

From the consideration of yield strength, gross area of the angle required $= \dfrac{225 \times 1000}{\left(\dfrac{250}{1.1}\right)} = 990 \text{ mm}^2$

Try ISA 10075, 8 mm which has gross area $A_g = 1336 \text{ mm}^2$.

Number of bolts required:

$$d = 20 \text{ mm} \therefore d_0 = 22 \text{ mm}$$

Use gusset plate of thickness 10 mm.

Strength of one bolt in single shear

$$= \frac{\frac{400}{\sqrt{3}}\left(0 + \frac{\pi}{4} \times 20^2 \times 0.78\right)}{1.25} = 45272 \text{ N}$$

Adopting edge distance $e = 40$ mm; pitch $p = 60$ mm,

K_b is smaller of $\dfrac{40}{3\times22}, \dfrac{60}{3\times22}-0.25, \dfrac{400}{410}, 1.0$ i.e., $K_b = 0.606$.

$$\therefore \quad V_{dpb} = \frac{1}{1.25}\times2.5\times0.606\times20\times8\times400 = 77568 \text{ N}$$

$\therefore$ Bolt value = 45272 N.

Number of bolts required $= \dfrac{225\times1000}{45272} \approx 5$

Provide the bolts as shown in Fig. 5.9.

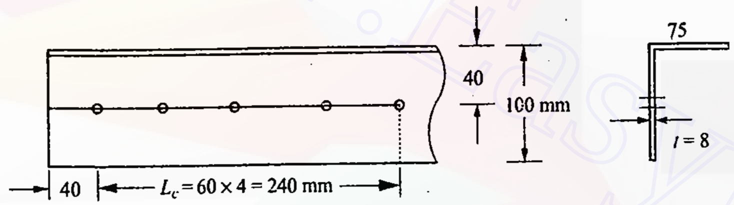


Figure 5.9

Checking the design:

(a) Strength against yielding:

$$= \frac{A_g f_y}{\gamma_{mo}} = \frac{1336\times250}{1.1} = 303636 \text{ N} > 225000 \text{ N} \qquad \textbf{O.K.}$$

(b) Strength of plate in rupture:

Area of connected leg $A_{nc} = \left(100-22-\dfrac{8}{2}\right)\times8 = 592 \text{ mm}^2$

Area of outstanding leg $A_{go} = \left(75-\dfrac{8}{2}\right)\times8 = 568 \text{ mm}^2$

$$\beta = 1.4 - 0.076\frac{75}{8}\times\frac{250}{410}\times\frac{(75+40-8)}{(40+4\times60-4\times22)}$$

$$= 1.094$$

$$\therefore \quad T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{ml}} + \beta\frac{A_{go} f_y}{\gamma_{mo}}$$

$$= \frac{0.9\times410\times592}{1.25} + 1.206\times\frac{568\times250}{1.1}$$

$$= 330442 \text{ N} > 225000 \text{ N}$$

O.K.

(c) Strength against block shear failure

$A_{vg} = (40+60\times4)\,8 = 2240 \text{ mm}^2$ $\quad A_{vn} = (40+60\times4-4.5\times22)\times8 = 1448 \text{ mm}^2$

$A_{tg} = (100-40)\times8 = 480 \text{ mm}^2$ $\quad A_{tn} = (100-40-0.5\times22)\times8 = 392 \text{ mm}^2$

Strength against block shear failure is smaller of

$$= \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{2240\times250}{\sqrt{3}\times1.1} + \frac{0.9\times392\times410}{1.25} = 409642 \text{ N}$$

$$\text{and} \quad \frac{0.9\,A_{vn} f_u}{\sqrt{3}\,\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9\times1448\times410}{\sqrt{3}\times1.25} + \frac{480\times250}{1.1}$$

$$= 355879 \text{ N}$$

$$\therefore \quad T_{db} = 355879 \text{ N} > 225000 \text{ N}.$$

O.K.

Hence safe.

Check for maximum $\dfrac{l}{r}$ **:** r- least radius of gyration = 12.7 mm (from steel table)

$$\therefore \quad \frac{l}{r} = \frac{3000}{12.7} = 236 < 350$$

Hence **O.K.**

Example 5.7

Design a double angle tension member connected on each side of a 10 mm thick gusset plate, to carry an axial factored load of 375 kN. Use 20 mm black bolts. Assume shop connection.

Solution:

Area required from the consideration of yielding $= \dfrac{1.1 \times 375 \times 1000}{250} = 1650 \text{ mm}^2$.

Try 2 ISA 7550, 8 mm thick which has gross area $= 2 \times 938 = 1876 \text{ mm}^2$.

Strength of 20 mm black bolts:

(a) In double shear $= \left[\dfrac{\pi}{4} \times 20^2 + 0.78 \times \dfrac{\pi}{4} \times 20^2\right] \times \dfrac{400}{\sqrt{3}} \dfrac{1}{1.25}$

$= 103314$ N.

(b) Strength in bearing:

Taking $e = 40$ mm, $p = 60$ mm,

K_b is smaller of $\dfrac{40}{3 \times 22}, \dfrac{60}{3 \times 22} - 0.25, \dfrac{400}{410}, 1.0$

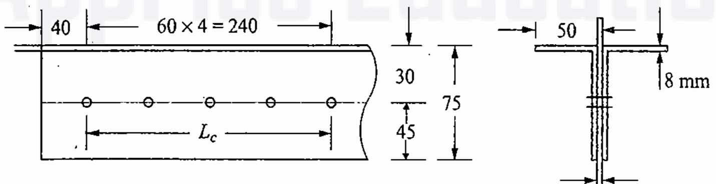
i.e., $K_b = 0.606$

$$\therefore \quad V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.606 \times 20 \times 8 \times 400 = 77568 \text{ N}$$

$\therefore$ Bolt value = 77568 N

Number of bolts required $= \dfrac{375000}{77568} = 4.83$

Provide 5 bolts in a row as shown in Fig. 5.10.



$w = 50$ mm

$b_s = 50 + 35 - 8 = 77$ mm

$L_c = 60 \times 4 = 240$ mm

Figure 5.10

Checking the design:

(a) Strength against yielding $= \dfrac{A_g f_y}{\gamma_{mo}} = \dfrac{1876 \times 250}{1.1} = 426364 \text{ N} > 375 \times 1000$

O.K

(b) Strength of plate in rupture:

Area of connected leg,

$$A_{nc} = 2\left(75 - 22 - \frac{8}{2}\right) \times 8 = 784 \text{ mm}^2$$

Area of outstanding leg,

$$A_{go} = 2 \times \left(50 - \frac{8}{2}\right) \times 8$$

$$= 736 \text{ mm}^2$$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{50}{8} \times \frac{250}{410} \times \frac{77}{240}$$

$$= 1.307$$

$$\therefore \quad T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{ml}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 410 \times 784}{1.25} + 1.307 \times \frac{736 \times 250}{1.1} = 450062 > 375000 \text{ N}$$

O.K.

(c) Strength against block shear failure:

Per angle:

$A_{vg} = (40 + 60 \times 4) \times 8 = 2240 \text{ mm}^2$
$A_{vn} = (40 + 60 \times 4 - 4.5 \times 22) \times 8 = 1448 \text{ mm}^2$
$A_{tg} = (75 - 35) \times 8 = 320 \text{ mm}^2$
$A_{tn} = (75 - 35 - 0.5 \times 22) \times 8 = 232 \text{ mm}^2$

Strength against block failure of each angle is the smaller of the following two values:

(i)
$$= \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 \times A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$$

$$= 362410 \text{ N}$$

(ii)
$$= \frac{0.9 A_{vn} f_u}{\sqrt{3} \times \gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.1}$$

$$= 319515 \text{ N}$$

$\therefore$ Strength of two angles against block failure $= 2 \times 319515 > 375000$

O.K.

Hence use 2 ISA 7550, 8 mm with 5 bolts of 20 mm diameter.

5.3 TENSION MEMBER SPLICE

If a single piece of required length is not available tension members are spliced to transfer required tension from one piece to another. The strength of the splice plates and the bolts/weld connecting them should have strength at least equal to the design load. When tension members of different thicknesses are to be connected, filler plates may be used to bring the members in level. The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor (clause 10.3.3.3 in IS 800-2007)

$$\beta_{pk} = 1 - 0.0125\, t_{pk}$$

where t_{pk} = thickness of the thicker packing plate.

Example 5.8

Design a splice to connect a 300 × 20 mm plate with a 300 × 10 mm plate. The design load is 500 kN. Use 20 mm black bolts, fabricated in the shop.

Solution:

Let double cover butt joint with 6 mm cover plates be used.

Strength of Bolts:

$d = 20$ mm, $d_0 = 22$ mm, $\beta_{pk} = 1 - 0.0125 \times 10 = 0.875$

Strength in double shear $= \beta_{pk}\left(\frac{\pi}{4} d^2 + 0.78 \frac{\pi}{4} d^2\right) \frac{f_u}{\sqrt{3} \times 1.25}$

$$= 0.875 \times 1.78 \times \left(\frac{\pi\, 20^2}{4}\right) \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25}$$

$$= 90400 \text{ N}$$

Strength in bearing:

Let edge distance = 40 mm and pitch 60 mm be used. Then K_b is smaller of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0.$

i.e., $\frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1.0$ i.e., $K_b = 0.606$

$\therefore$ Strength in bearing against 10 mm plate

$$= 2.5\, K_b\, d t f_u \times \frac{1}{\gamma_{mb}}$$

$$= 2.5 \times 0.606 \times 20 \times 10 \times 410 \times \frac{1}{1.25}$$

$$= 99384 \text{ N} < 105897 \text{ N}$$

$\therefore$ Bolt value = 90400 N

Hence number of bolts required $= \frac{500 \times 1000}{90400} = 5.53$

Provide 6 bolts on each side of the joint as shown in Fig. 5.11

Check for the strength of plate:

(i) Strength against yielding of gross section

$$= \frac{A_g f_y}{\gamma_{mo}} = \frac{300 \times 10 \times 250}{1.1} = 681818 \text{ N} > 50000 \text{ N}$$

(ii) Strength against rupture:

$$A_n = (300 - 3 \times 22) \times 10 = 2340 \text{ mm}^2$$

$$\therefore \quad T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} = \frac{0.9 \times 2340 \times 410}{1.25}$$

$$= 690768 \text{ N} > 500000 \text{ N}$$

O.K.

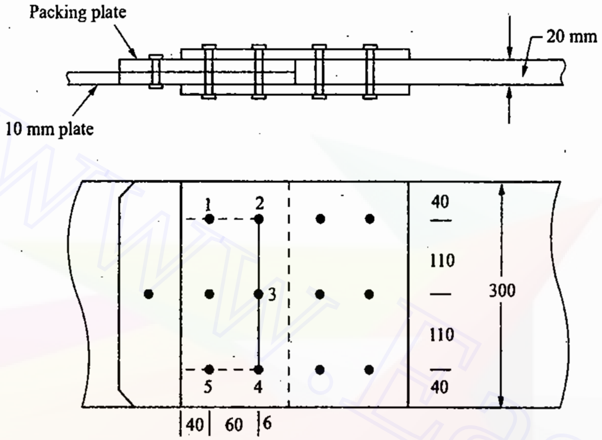


Figure 5.11

(iii) Block shear strength

(a) Along 1-1-2-4-5-1:

$A_{vg} = 2\,(40 + 60) \times 10 = 2000 \text{ mm}^2$

$A_{vn} = 2\,(40 + 60 - 1.5 \times 22) \times 10 = 1340 \text{ mm}^2$

$A_{tg} = 220 \times 10 = 2200 \text{ mm}^2$

$A_{tn} = (220 - 2 \times 22) \times 10 = 1760 \text{ mm}^2$

$$\therefore T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{2000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1760 \times 410}{1.25} = 781984 \text{ N}$$

$$\text{or } T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 1340 \times 410}{\sqrt{3}\,1.25} + \frac{2200 \times 250}{1.1} = 728381 \text{ N}$$

(b) Along 1-1-2-3-4-6-1:

$A_{vg} = (40 + 60) \times 10 = 1000 \text{ mm}^2$

$A_{vn} = (40 + 60 - 1.5 \times 22) \times 10 = 670 \text{ mm}^2$

$A_{tg} = (220 + 40) \times 10 = 2600 \text{ mm}^2$

$A_{tn} = (260 - 2 \times 22) \times 10 = 2160 \text{ mm}^2$

$$\therefore T_{db} = \frac{1000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2160 \times 410}{1.25} = 768848 \text{ N}$$

$$\text{or} \quad T_{db} = \frac{0.9 \times 670 \times 410}{\sqrt{3} \times 1.25} + \frac{2600 \times 250}{1.1} = 705100 \text{ N}$$

$\therefore$ Block shear strength is 705100 N > 50000 N

Hence O.K.

Provide an extra bolt in the caver plate on packing material [Ref. Fig 5.11]

5.4 LUG ANGLES

Length of the end connection of a heavily loaded tension member may be reduced by using lug angles as shown in Fig. 5.12. By using lug angles there will be saving in gusset plate, but it is upset by additional fasteners and angle required. Hence nowadays it is not preferred. IS 800-2007 specifications for lug angles are (clause 10.12)

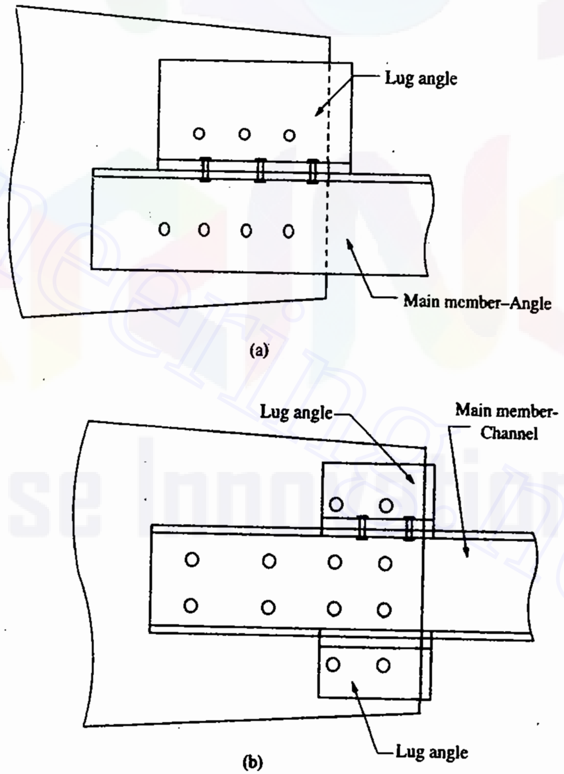


Figure 5.12

1. The effective connection of the lug angle shall as far as possible terminate at the end of the member.
2. The connection of lug angle to main member shall preferably start in advance of the member to the gusset plate.
3. Minimum of two bolts, rivets or equivalent welds be used for attaching lug angle to the gusset.
4. If the main member is an angle
   (a) the whole area of the member shall be taken as the effective rather than net effective section (i.e., with reduction for outstanding leg area). The whole area of the member is the gross area less deduction for bolt holes.
   (b) the strength of lug angles and fastener connecting lug angle to gusset plate should be at least 20 percent more than the force in outstanding leg.
   (c) the strength of the fastener connecting lug angle and main member shall be at least 40% more than the force carried by the outstanding leg.
5. In case the main member is a channel and like:
   (a) as far as possible should be symmetric.
   (b) the strength of fasteners connecting lug angle to the gusset should be at least 10% more than the force in outstanding leg.
   (c) the strength of fasteners connecting lug angle and main member shall be at least 20% more than the force in outstanding leg.

Example 5.9

A tension member of a roof truss carries a factored axial tension of 430 kN. Design the section and its connection

(a) without using lug angle
(b) using lug angle.

Solution:

Tensile force in the main member = 430 kN. Considering the strength in yield, gross area required is given by, $430 \times 1000 = \frac{A_g f_y}{1.1} = \frac{A_g \times 250}{1.1}$

$$A_g = 1892 \text{ mm}^2$$

Select ISA 100100, 10 mm which has

$$A_g = 1903 \text{ mm}^2$$

Using 20 mm diameter black bolts,

Strength in single shear:

$$T_{dg} = 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 45272 \text{ N}$$

Strength in bearing:

$e_{min} = 1.5 \times 20 = 30$ mm $\quad p_{min} = 2.5 \times 20 = 50$ mm

Let $e = 30$ mm $\quad p = 50$ mm

Then K_b is smaller of $\frac{30}{3 \times 22}, \; \frac{50}{2 \times 22} - 0.25, \; \frac{400}{410}, \; 1.0$

$$\therefore \; K_b = 0.4545$$

$$\therefore \; T_{dn} = 2.5 \times 0.4545 \times 20 \times 10 \times \frac{400}{1.25} = 72720 \text{ N.}$$

$\therefore$ Bolt value = 45272 N.

Note: In case of single shear, bolt value is usually governed by value in single shear.

(a) Connection without lug angle:

Number of bolts required $= \frac{430000}{45270} = 9.5$

Provide 10 bolts.

Length of connection, $L_c = 9 \times 50 = 450$ mm

$15\,d = 15 \times 20 = 300$ mm.

$\therefore$ $L_c > 15\,d$. It is long connection.

$\therefore$ $\beta_{lj} = 1.075 - 0.005 \, \frac{450}{20} = 0.9625$

Shear strength of bolt (after reducing for long connection)
$= 0.9625 \times 45272 = 435743$ N

$\therefore$ No. of bolts required $= \frac{430000}{43574.3} = 9.87$

Hence 10 bolts are sufficient.

Yield strength $= \frac{A_g f_y}{1.1} = \frac{1903 \times 250}{1.1} = 432500 \text{ N} > 430 \times 10^3$ N. **Hence O.K.**

Rupture strength:

$$A_{nc} = \left(100 - \frac{10}{2} - 22\right) \times 10 = 730 \text{ mm}^2$$

$$A_{go} = \left(100 - \frac{10}{2}\right) \times 10 = 950 \text{ mm}^2$$

$$\beta = 1.4 - 0.076 \left(\frac{95}{10}\right) \times \frac{250}{410} \times \frac{130}{450} = 1.2728 > 0.7 \text{ and } < \frac{f_u}{f_y}\frac{\gamma_{mo}}{\gamma_{ml}}$$

$\therefore \beta = 1.2728$

Strength in rupture $= \dfrac{730 \times 0.9 \times 410}{1.25} + \dfrac{1.2728 \times 950 \times 250}{1.1} = 490305$ N

Block shear strength:

$A_{vg} = (450 + 30) \times 10 = 4800 \text{ mm}^2$, $A_{tg} = 70 \times 10 = 700 \text{ mm}^2$

$A_{vn} = (480 - 9.5 \times 22) \times 10 = 2710 \text{ mm}^2$, $A_{tn} = \left(70 - \dfrac{22}{2}\right) \times 10 = 490 \text{ mm}^2$

$\therefore$ Block shear strength $= \dfrac{4800 \times 250}{\sqrt{3} \times 1.1} + \dfrac{490 \times 0.9 \times 410}{1.25} = 774485$ N

or $\dfrac{0.9 \times 2710 \times 410}{\sqrt{3} \times 1.25} + \dfrac{700 \times 0.9 \times 250}{1.1} = 605057$ N

$\therefore$ Block shear strength = 605057 N

Hence strength of angle is 432500 N > 430×10^3 N. **Hence O.K.**

(b) Connection with lug angle:

Gross area of connected leg = Gross area of outstanding leg

$\therefore$ Load is shared equally.

i.e., Load in outstanding leg = Load in connected leg $= \dfrac{430}{2} = 215$ kN.

Lug angle is to be designed to take a load of $= 1.2 \times 215 = 258$ kN.

Gross area of lug angle required $= \dfrac{258 \times 1000}{250/1.1} = 1135 \text{ mm}^2$

Provide ISA 100100, 6 mm.

$\therefore A_g$ provided = 1167 mm^2

The strength of lug angle in rupture $= \dfrac{0.9 \times (100 + 100 - 10 - 22) 6 \times 410}{1.25}$

$= 297562 \text{ N} > 258000$ N. O.K

Bolt value:

In single shear = 45272 N

In bearing $= \dfrac{2.5 \times 0.4545 \times 20 \times 6 \times 400}{1.25} = 43632$

$\therefore$ Bolt value = 43630 N

Number of bolts required $= \dfrac{258 \times 1000}{43630} = 5.91$

Provide 6 bolts.

Design force for connected leg $= 1.4 \times 215$ kN

$\therefore$ Number of bolts required to connect lug angle with main angle $= \dfrac{1.4 \times 215 \times 1000}{43630} = 6.89$

Provide 7 bolts.

Connection of main angle to gusset plates:

Force to be transferred = 215 kN

Bolt value for this is 45272 N.

$\therefore$ No. of bolts required $= \dfrac{215000}{45270} = 4.75$

Provide 5 bolts.

Required length of gusset plate

$= 30 + (7 - 1) \times 50 = 330$ mm (compared to 480 mm required without lug angle)

[Block shear strength may be checked. It is safe.]

The connection detail is shown in the Fig. 5.13.

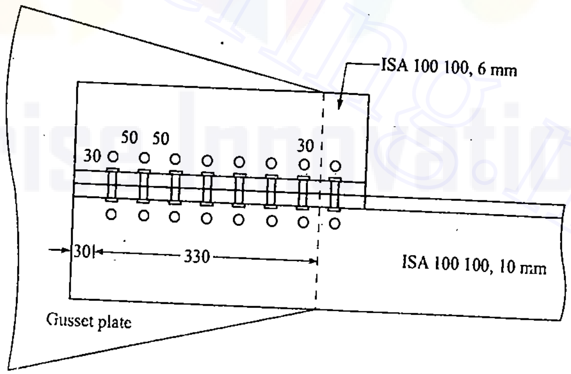


Figure 5.13

* * *

Questions

1. Explain the different modes of failure of tension members.
2. Write short note on block shear failure.
3. What is a lug angle? Illustrate with sketch. Why lug angles are used?
4. Write short notes on tension member splices.
5. Determine the tensile strength of the plate 160 mm × 10 mm with the holes for 24 mm bolts as shown in Fig. 5.14.
6. Determine the tensile strength of a roof truss diagonal 100 × 75 × 10 mm. The longer leg is connected to the gusset plate with 20 mm diameter bolts in one row. Number of bolts used is 6, the edge/end distance = 30 mm and pitch = 50 mm.
7. A member consists of a single angle ISA 150 × 75. It is to be connected to the gusset plate by two rows of 20 mm diameter bolts at a pitch of 80 mm with a stagger of 40 mm. The first line of bolt is located at their centres 50 mm from the back of the angle while the second row is located at 60 mm from the first row. The tensile force (working) is 200 kN. Calculate the thickness of angle.
8. Design a tension member to carry a factored force of 340 kN. Use 20 mm diameter black bolts and a gusset plate of 8 mm thick.
9. A tension member of truss consists of a single angle ISA 125 × 75 × 10 mm carrying a factored load of 300 kN, if 20 mm diameter bolts are used. Design the connection to a gusset plate using a lug angle.

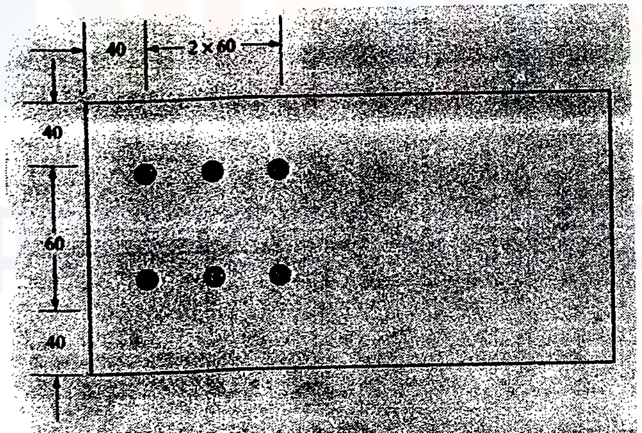


Figure 5.14

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6

DESIGN OF COMPRESSION MEMBERS

Many structural members are in compression. Vertical compression members in buildings are called columns, posts or stanchions. Compression members in trusses are called struts. The jib of crane which carries compression is called boom.

Whatever care taken by the engineers to transfer load axially unexpected eccentricity of load is unavoidable due to imperfection. This eccentricity causes lateral bending moment which results into bending compression also. As the axial compression increases the lateral deflection increases resulting into additional bending stresses. A stage of instability is reached at a load much below crushing strength of compression members. This phenomenon is called buckling of columns. Because of buckling tendency the load carrying capacity of columns is reduced considerably. The load carrying capacity depends upon the end conditions and also on slenderness ratio of the column sections.

In this chapter different buckling classes based on possible imperfections in the column is discussed. Then the method of determining effective length depending upon end conditions is presented. Finally, finding design stresses in compression members based on slenderness ratio, as accepted by IS 800-2007 is illustrated. Many examples are solved to illustrate design of different compression members and method of splicing them.

6.1 BUCKLING CLASS OF CROSS-SECTION

Imperfections of fabrication resulting into accidental eccentricity largely depends upon the cross-section of the compression members. Based on such imperfection buckling tendency varies. IS 800-2007 divides various cross-sections into four buckling classes a, b, c and d as shown in Table 6.1. (Table 10 in IS 800)

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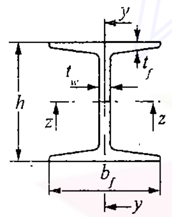
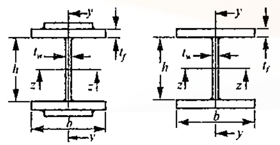
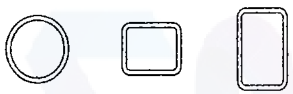
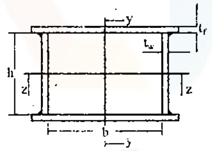
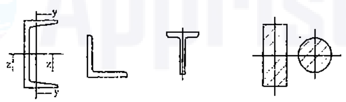
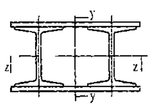
Table 6.1 Buckling class of cross-sections
[Refer Table 10 in IS 800]

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	40 mm $< t_f \le$ 100 mm	z-z y-y	b c
	$\frac{h}{b_f} \le 1.2$ $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section 	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, *T* and Solid Sections 		Any	c
Built-up Member 		Any	c

6.2 SLENDERNESS RATIO

Slenderness ratio of a column is defined as the ratio of effective length to corresponding radius of gyration of the section. Thus

$$\text{slenderness ratio} = \frac{l_e}{r} = \frac{KL}{r}$$

where, L = actual length of compression member

$l_e = KL$, effective length

r = appropriate radius of gyration.

6.2.1 Actual Length

It is the centre to centre distance of compression member between the restrained ends. In Fig. 6.1, 6 m column is restrained at ends *A* and *B* in both *y-y* and *z-z* direction. At *C* it is restrained in *z-z* direction only. Hence its actual length in *y-y* direction is 6 m while in *z-z* direction it is equal to *AC* = 3 m only.

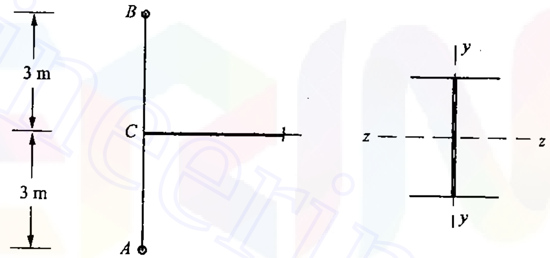


Figure 6.1

6.2.2 Effective Length

The effective length *KL* is calculated from the actual length *L*, of the member considering the rotational and relative translational boundary conditions at the ends. IS 800-2007 recommends the following:

(a) If end conditions can be assessed:

Where the boundary conditions in plane of buckling can be assessed the effective length *KL* can be calculated on the bases of Table 6.2 (Refer Table 11 in IS 800).

(b) Compression members in trusses:

(i) In the case of bolted, riveted or welded trusses and braced frames, the effective length, KL, shall be taken as 0.7 to 1.0 times the actual length, depending upon the degree of end restraints provided.

(ii) For buckling in the plane perpendicular to the plane of truss, the effective length may be taken as actual length.

(c) In frames:
In the frame analysis, if deformed shape is not considered (second order or advanced analysis is not used), the effective length depends upon stiffnesses of the members meeting at the joint. The method of finding effective length factor K are shown in Annex D of IS 800. One can use the graphs given in the annexure.

(d) In case of stepped columns:
Expressions for finding effective length factor for various stepped columns are presented in IS 800 annexure D2 and D3.

6.2.3 Appropriate Radius of Gyration

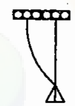
Appropriate radius of gyration means the radius of gyration of compression member about the axis of buckling. For example, in case of column shown in Fig. 6.1, when length of the column is taken 6 m, the radius of gyration about z-z axis should be considered. For buckling about y-y axis, the length of column is 3 m and radius of gyration about y-y axis is to be considered. The maximum slenderness ratio governs the design strength. If the length of the column to be considered is the same for buckling about any axis, naturally the governing slenderness ratio is $\dfrac{KL}{r_{\min}}$.

6.3 DESIGN COMPRESSIVE STRESS AND STRENGTH

The design compressive stress, f_{cd} of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}} \le \frac{f_y}{\gamma_{mo}}$$

Table 6.2 Effective length of prismatic compression members
[Refer Table 11 in IS 800]

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation (1)	Rotation (2)	Translation (3)	Rotation (4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Restrained	Free	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

Note: L is the unsupported length of the compression member.

where $\phi = 0.5\,[1 + \alpha(\lambda - 0.2) + \lambda^2]$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y\left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

f_{cc} = Euler buckling stress

α = imperfection factor, given in Table 6.3

γ_{mo} = 1.1 for Fe 415 steel.

The design compressive strength P_d of a member is given by

$$P_d = A_e f_{cd}$$

where A_e effective sectional area, which is the same as gross area if bolt holes are filled with bolts. Deductions for bolt holes may be made only if the holes are not fitted with bolts.

Table 6.3 Imperfection factor: α

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76

Example 6.1

Determine the design axial load capacity of the column ISHB 300 @ 577 N/m if the length of column is 3 m and its both ends pinned.

Solution:

For rolled steel sections,

$f_y = 250$ N/mm^2, $f_u = 410$ N/mm^2 and $E = 2 \times 10^5$ N/mm^2.

For both end pinned columns,

$KL = L = 3$ mm.

For ISHB 300 @ 577 N/m.

$h = 300$ mm, $b_f = 250$ mm, $t_f = 10.6$ mm, $A_e = A = 7484$ mm^2

$\therefore \dfrac{h}{b_f} = 1.2$ and $t_f < 40$ mm.

Hence according to Table 6.1 (Refer Table 10 in IS 800)

it falls under buckling class 'b' for buckling about *z-z* axis and under class 'c' for buckling about *y-y* axis.

From steel table $r_{min} = r_{yy} = 54.1$ mm.

$$\therefore \quad f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{3000}{54.1}\right)^2} = 641.92 \text{ N/mm}^2$$

Non-dimensionalised effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.92}} = 0.624$$

For buckling class b,

$\alpha = 0.34$.

$$\therefore \quad \phi = 0.5\,[1 + \alpha(\lambda - 0.2) + \lambda^2]$$

$$= 0.5\,[1 + 0.34\,(0.624 - 0.2) + 0.624^2]$$

$$= 0.767$$

$$\therefore \quad f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}}$$

$$= \frac{250/1.1}{0.767 + \left(0.767^2 - 0624^2\right)^{0.5}}$$

$$= 187.36 \text{ N/mm}^2$$

$\therefore$ Strength of column

$$P_d = A_c f_{cd} = 7484 \times 187.36$$

$$= 1402237 \text{ N}$$

$$= 1402.237 \text{ kN}$$

$\therefore$ Working load $= \dfrac{1402.237}{1.5} = 934.823$ kN **Answer**

6.4 I.S. TABLES FOR DESIGN STRESS

For the benefit of users tables are given in IS 800-2007 (Refer Table 9) to find design stress f_{cd}, if $\dfrac{KL}{r}$ is determined for all the four (*a*, *b*, *c* and *d*) classes of buckling. The tables are reproduced here for the benefit of readers of this book (Table 6.4). It may be verified that in the above problem $\dfrac{KL}{r} = \dfrac{3000}{51.8} = 57.9$ and from the Table 6.4(c), $f_{cd} = 171.4$.

Table 6.4(a) Design compressive stress, f_{cd} (MPa) for column buckling class a (Refer Table 9(a) in IS 800)

KL/r ↓	Yield Stress, f_y (MPa)																	
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	213	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	191	200	208	217	226	235	252	270	287	305	322	339	357	374	400	425	451	476
30	178	186	195	203	212	220	229	245	262	279	295	311	328	344	360	384	408	431	454
40	173	181	189	197	205	213	221	237	253	269	283	298	313	328	342	363	384	405	425
50	168	176	183	191	198	205	213	227	241	255	268	281	294	306	318	336	352	368	383
60	162	169	175	182	189	195	202	214	226	237	248	258	268	278	286	299	310	320	329
70	154	160	166	171	177	182	188	197	207	215	223	230	237	243	249	256	263	268	274
80	144	149	154	158	163	167	171	178	184	190	195	199	204	207	210	215	219	222	225
90	133	137	140	143	146	149	152	157	161	164	168	170	173	175	177	179	182	184	185
100	120	123	125	128	130	132	133	136	139	141	143	145	146	148	149	151	152	153	154
110	107	109	111	112	114	115	116	118	120	121	123	124	125	126	127	128	129	129	130
120	95.5	96.7	97.9	98.9	100	101	101	103	104	105	106	107	107	108	109	109	110	110	111
130	84.6	85.5	86.3	87.0	87.7	88.3	88.8	89.8	90.6	91.3	92.0	92.5	93.0	93.5	93.9	94.4	94.9	95.3	95.7
140	75.2	75.8	76.4	76.9	77.4	77.8	78.2	78.9	79.5	80.0	80.5	80.9	81.3	81.6	81.9	82.3	82.6	83.0	83.2
150	67.0	67.4	67.9	68.2	68.6	68.9	69.2	69.7	70.2	70.6	70.9	71.2	71.5	71.8	72.0	72.3	72.6	72.9	73.1
160	59.9	60.3	60.6	60.9	61.1	61.4	61.6	62.0	62.4	62.7	62.9	63.2	63.4	63.6	63.8	64.0	64.3	64.5	64.6
170	53.8	54.1	54.3	54.6	54.8	55.0	55.1	55.5	55.7	56.0	56.2	56.4	56.6	56.7	56.9	57.1	57.3	57.4	57.6
180	48.6	48.8	49.0	49.2	49.3	49.5	49.6	49.9	50.1	50.3	50.5	50.6	50.8	50.9	51.0	51.2	51.3	51.5	51.6
190	44.0	44.2	44.3	44.5	44.6	44.7	44.9	45.1	45.3	45.4	45.6	45.7	45.8	45.9	46.0	46.2	46.3	46.4	46.5
200	40.0	40.2	40.3	40.4	40.5	40.7	40.7	40.9	41.1	41.2	41.3	41.4	41.5	41.6	41.7	41.8	41.9	42.0	42.1
210	36.6	36.7	36.8	36.9	37.0	37.1	37.2	37.3	37.4	37.6	37.7	37.8	37.8	37.9	38.0	38.1	38.2	38.3	38.3
220	33.5	33.6	33.7	33.8	33.9	34.0	34.0	34.2	34.3	34.4	34.5	34.5	34.6	34.7	34.7	34.8	34.9	35.0	35.0
230	30.8	30.9	31.0	31.1	31.2	31.2	31.3	31.4	31.5	31.6	31.6	31.7	31.8	31.8	31.9	31.9	32	32.1	32.1
240	28.5	28.5	28.6	28.7	28.7	28.8	28.8	28.9	29.0	29.1	29.1	29.2	29.3	29.3	29.4	29.4	29.5	29.5	29.6
250	26.3	26.4	26.5	26.5	26.6	26.6	26.7	26.7	26.8	26.9	26.9	27.0	27.0	27.1	27.1	27.2	27.2	27.3	27.3

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Table 6.4(b) Design compressive stress, f_{cd} (MPa) for column buckling class b (Refer Table 9(b) in IS 800)

KL/r ↓	Yield Stress, f_y (MPa)																	
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	208	217	225	234	251	268	285	302	319	336	353	369	394	419	443	468
30	175	183	192	200	208	216	224	240	256	271	287	302	318	333	348	370	392	414	435
40	169	176	183	191	198	206	213	228	242	256	270	283	297	310	323	342	360	378	395
50	161	167	174	181	188	194	201	214	226	238	250	261	272	283	293	308	322	335	347
60	152	158	164	170	176	181	187	197	207	217	226	235	243	251	259	269	279	287	295
70	142	147	152	157	162	166	171	179	187	194	201	207	213	218	223	230	236	241	246
80	131	135	139	143	147	150	154	160	165	170	175	179	183	186	190	194	190	201	204
90	120	123	126	129	131	134	136	141	144	148	151	154	156	159	161	163	166	168	170
100	108	110	112	114	116	118	120	123	126	128	130	132	134	135	137	139	140	142	143
110	96.5	98.3	100	101	103	104	105	107	109	111	112	114	115	116	117	118	119	121	121
120	86.2	87.5	88.6	89.7	90.7	91.7	92.5	94.1	95.4	96.6	97.7	98.6	100	100	101	102	103	104	104
130	76.9	77.8	78.7	79.5	80.3	81.0	81.6	82.7	83.7	84.6	85.4	86.1	86.8	87.3	87.9	88.6	89.2	89.8	90.3
140	68.7	69.4	70.1	70.7	71.3	71.8	72.3	73.1	73.9	74.6	75.2	75.7	76.2	76.6	77.1	77.6	78.1	78.5	78.9
150	61.6	62.1	62.6	63.1	63.6	64.0	64.3	65.0	65.6	66.1	66.6	67.0	67.4	67.7	68.1	68.5	68.9	69.2	69.5
160	55.4	55.8	56.2	56.6	56.9	57.3	57.5	58.1	58.5	59.0	59.3	59.7	60.0	60.3	60.5	60.9	61.2	61.5	61.7
170	50.0	50.3	50.7	51.0	51.2	51.5	51.7	52.2	52.5	52.9	53.2	53.5	53.7	53.9	54.1	54.4	54.7	54.9	55.1
180	45.3	45.6	45.9	46.1	46.3	46.5	46.7	47.1	47.4	47.7	47.9	48.1	48.3	48.5	48.7	48.9	49.2	49.3	49.5
190	41.2	41.5	41.7	41.9	42.1	42.2	42.4	42.7	42.9	43.2	43.4	43.6	43.7	43.9	44.0	44.2	44.4	44.6	44.7
200	37.6	37.8	38.0	38.2	38.3	38.5	38.6	38.9	39.1	39.3	39.5	39.6	39.8	39.9	40.0	40.2	40.3	40.5	40.6
210	34.5	34.7	34.8	35.0	35.1	35.2	35.3	35.5	35.7	35.9	36.0	36.2	36.3	36.4	36.5	36.6	36.8	36.9	37.0
220	31.7	31.9	32.0	32.1	32.2	32.3	32.4	32.6	32.8	32.9	33.0	33.1	33.2	33.3	33.4	33.6	33.7	33.8	33.9
230	29.2	29.4	29.5	29.6	29.7	29.8	29.9	30.0	30.1	30.3	30.4	30.5	30.6	30.7	30.7	30.8	30.9	31.0	31.1
240	27.1	27.2	27.3	27.3	27.4	27.5	27.6	27.7	27.8	27.9	28.0	28.1	28.2	28.3	28.3	28.4	28.5	28.6	28.7
250	25.1	25.2	25.3	25.3	25.4	25.5	25.6	25.7	25.8	25.9	26.0	26.0	26.1	26.2	26.2	26.3	26.4	26.5	26.5

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Table 6.4(c) Design compressive stress, f_{cd} (MPa) for column buckling class c
(Refer Table 9(c) in IS 800)

KL/r ↓	Yield Stress, f_y (MPa) 200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93	93.8	94.9	95.9	96.8	97.6
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7

Table 6.4(d) Design compressive stress, f_{cd} (MPa) for column buckling class d
(Refer Table 9(d) in IS 800)

KL/r ↓	Yield Stress, f_y (MPa) 200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	198	206	215	223	231	247	263	279	294	310	325	340	355	377	399	421	442
30	168	175	182	189	197	204	211	224	238	251	264	277	290	302	314	332	350	367	384
40	154	161	167	173	179	185	191	203	214	225	235	246	256	266	275	289	303	316	328
50	141	147	152	157	162	167	172	182	191	199	208	216	224	231	238	249	258	268	277
60	129	133	137	142	146	150	154	161	168	175	182	188	193	199	204	212	219	225	231
70	116	120	124	127	130	133	137	142	148	153	158	162	167	171	174	180	184	189	193
80	105	108	111	113	116	118	121	125	129	133	137	140	143	146	149	153	156	159	162
90	94.1	96.4	98.6	101	103	105	107	110	113	116	119	121	123	126	128	130	133	135	137
100	84.3	86.2	87.9	89.6	91.1	92.6	94.0	96.7	99.1	101	103	105	107	108	110	112	114	116	117
110	75.6	77.0	78.4	79.7	81.0	82.1	83.2	85.3	87.1	88.8	90.4	91.8	93.1	94.4	95.5	97.1	98.5	100	101
120	67.8	69.0	70.1	71.1	72.1	73.0	73.9	75.5	77.0	78.3	79.5	80.6	81.7	82.6	83.5	84.7	85.8	86.9	87.8
130	61.0	62.0	62.8	63.7	64.5	65.2	65.9	67.2	68.3	69.4	70.4	71.2	72.1	72.8	73.5	74.5	75.4	76.2	76.9
140	55.0	55.8	56.5	57.2	57.8	58.4	59.0	60.0	61.0	61.8	62.6	63.3	64.0	64.6	65.2	66.0	66.7	67.3	67.9
150	49.8	50.4	51.0	51.6	52.1	52.6	53.1	53.9	54.7	55.4	56.0	56.6	57.2	57.7	58.1	58.8	59.3	59.9	60.4
160	45.2	45.7	46.2	46.7	47.1	47.5	47.9	48.6	49.3	49.9	50.4	50.9	51.3	51.7	52.1	52.7	53.1	53.6	54.0
170	41.2	41.6	42.1	42.4	42.8	43.1	43.5	44.1	44.6	45.1	45.5	45.9	46.3	46.7	47.0	47.4	47.8	48.2	48.6
180	37.7	38.0	38.4	38.7	39.0	39.3	39.6	40.1	40.5	41.0	41.3	41.7	42.0	42.3	42.6	43.0	43.3	43.6	43.9
190	34.5	34.9	35.2	35.4	35.7	35.9	36.2	36.6	37.0	37.4	37.7	38.0	38.2	38.5	38.7	39.1	39.4	39.6	39.9
200	31.8	32.0	32.3	32.5	32.8	33.0	33.2	33.6	33.9	34.2	34.5	34.7	35.0	35.2	35.4	35.7	35.9	36.2	36.4
210	29.3	29.6	29.8	30.0	30.2	30.4	30.5	30.9	31.2	31.4	31.7	31.9	32.1	32.3	32.5	32.7	32.9	33.1	33.3
220	27.1	27.3	27.5	27.7	27.9	28.0	28.2	28.5	28.7	29.0	29.2	29.4	29.6	29.7	29.9	30.1	30.3	30.5	30.6
230	25.2	25.3	25.5	25.7	25.8	26.0	26.1	26.4	26.6	26.8	27.0	27.1	27.3	27.5	27.6	27.8	27.9	28.1	28.2
240	23.4	23.6	23.7	23.9	24.0	24.1	24.2	24.5	24.7	24.8	25.0	25.2	25.3	25.4	25.5	25.7	25.9	26.0	26.1
250	21.8	22.0	22.1	22.2	22.3	22.5	22.6	22.8	22.9	23.1	23.2	23.4	23.5	23.6	23.7	23.9	24.0	24.1	24.2

6.5 SHAPES OF COMPRESSION MEMBERS

Since the design stress in compression member decreases with the least radius of gyration, the section should be proportioned to have maximum moment of inertia for the same sectional area. This can be achieved by concentrating the area away from centroid of the section. As far as possible the section should have approximately the same radius of gyration about any axis. This requirement is fulfilled by circular tubes. Due to difficulties in making end connections, they were not commonly used earlier. But nowadays due to improvements in welding technology tubular sections are getting popularity as compression members. Next best shape may be square tubing. Among I-sections ISHB sections are preferable as columns, since they have better r_{min} values for the same area of cross-sections. If built up areas are required strengthening should be made by connecting plates on flanges so as to increase r_{zz} value which is lower compared to r_{yy} (Ref. Table 6.1) value. In roof trusses and transmission towers angle sections are commonly used. It is preferable to use equal angles instead of unequal angles as compression members since such angles have higher r_{min} values for the same cross sectional areas. Various shapes of commonly used compression members are shown in Fig. 6.2.

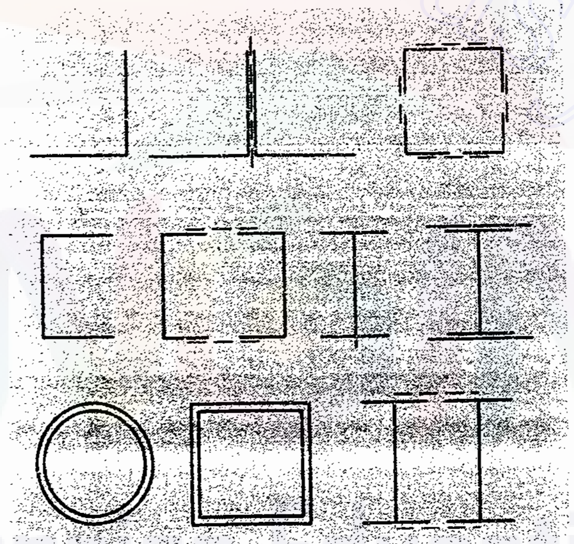

Figure 6.2

Example 6.2

In a truss a strut 3 m long consists of two angles ISA 100100, 6 mm. Find the factored strength of the member if the angles are connected on both sides of 12 mm gusset by

(i) one bolt
(ii) two bolts
(iii) Welding, which makes the joint rigid.

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Solution:

From steel table for a ISA 100100, 6 mm,

area = 1167 mm^2; $C_{zz} = C_{yy} = 26.7$ mm.

$r_{zz} = r_{yy} = 30.9$ mm.

Figure 6.3 shows the details of the member.

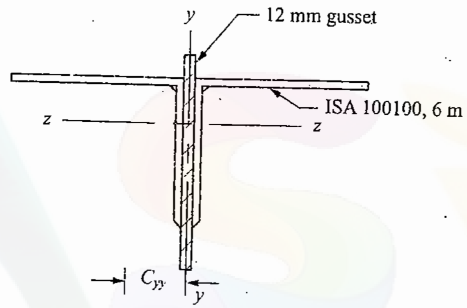

Figure 6.3

r_{zz} of the member is the same as r_{zz} of single angle, since the z-z axis for both is the same, resulting into doubling of I_{zz} and area.

$\therefore \quad r_{zz} = 30.9$ mm.

$I_{yy} = 2\,[I_{yy}$ of one angle + Area of one angle $\times (C_{yy} + 6)^2]$.

From steel table, I_{yy} of one angle $= 111.3 \times 10^4$

$$I_{yy} \text{ of the member } = 2\left[111.3\times10^4 + 1167\times(26.7+6)^2\right]$$

$$= 4721723 \text{ mm}^4$$

$$\therefore \quad r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4721723}{2\times1167}} = 44.98$$

$\therefore \quad r_{zz}$ is governing the strength of member.

Case (i): When a single bolt is used

$r = r_{zz} = 30.9$ mm

$KL = L = 3000$ mm.

$$\therefore \quad \frac{KL}{r} = \frac{3000}{30.9} = 97$$

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The member belongs to buckling class c (Ref. Table 6.4)

Hence referring to Table 6.4(c), for $\frac{KL}{r} = 97$, corresponding to $f_y = 250$ MPa,

$$f_{cd} = 121 - \frac{7}{10}(121 - 107)$$

$$= 111.2 \text{ N/mm}^2.$$

$$\therefore \quad P_d = A_e f_{cd}$$

$$= 2 \times 1167 \times 111.2 = 259541 \text{ N}.$$

i.e. $P_d = 259.541$ kN **Answer**

Case (ii): When two bolts are used

The effective length is reduced. It may be taken as 0.85 times actual length.

$$\therefore \quad KL = 0.85 \times 3000 = 2550 \text{ mm}.$$

Hence in this case $\frac{KL}{r} = \frac{2550}{30.9} = 82.5$

From Table 6.4(c), for steel with $f_y = 250$ N/mm^2,

f_{cd} for $\frac{KL}{r} = 80$ is 136 N/mm^2

for $\frac{KL}{r} = 90$ is 121 N/mm^2

$\therefore$ Linearly interpolating, f_{cd} for $\frac{KL}{r} = 82.5$ is

$$f_{cd} = 136 - \frac{2.5}{10} \times (136 - 121)$$

$$= 132.25 \text{ N/mm}^2$$

$$\therefore \quad P_d = 2 \times 1167 \times 132.25$$

$$= 308672 \text{ N} = 308.672 \text{ kN}$$ **Answer**

Case (iii): Rigid joint by welding

Effective length $KL = 0.7 \times L = 0.7 \times 3000 = 2100$ mm

$$\therefore \quad \frac{KL}{r} = \frac{2100}{30.9} = 67.96$$

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From the table, f_{cd} values are

for $\frac{KL}{r} = 60 \quad f_{cd} = 168$ N/mm^2

$\frac{KL}{r} = 70 \quad f_{cd} = 152$ N/mm^2

$$\therefore \text{ For } \frac{KL}{r} = 67.96, \quad f_{cd} = 168 - \frac{7.96}{10}(168 - 152)$$

$$= 155.26 \text{ N/mm}^2$$

$$\therefore \quad P_d = 2 \times 1167 \times 155.26 = 362386 \text{ N}$$

$$= 362.386 \text{ kN}$$ **Answer**

Example 6.3

Determine the load carrying capacity of the column section shown in Fig. 6.4, if its actual length is 4.5 m. Its one end may be assumed fixed and the other end hinged. The grade of steel is Fe 415 (i.e. E 250).

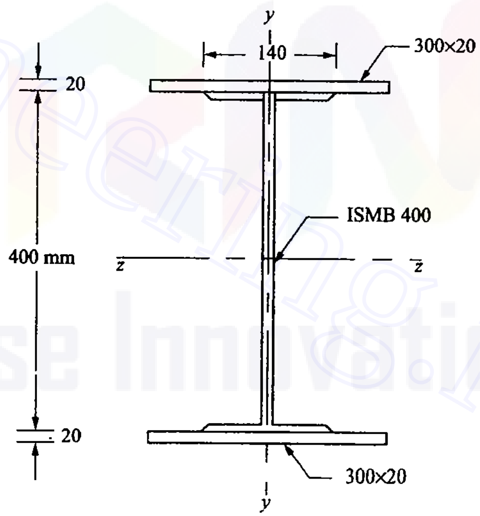

Figure 6.4

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Solution:

For ISMB 400,

$h = 400$ mm, $b_f = 140$ mm, $t_f = 16$ mm, $I_{zz} = 20458.4 \times 10^4$ mm^4

$I_{yy} = 622.1 \times 10^4$ mm^4, Area = 7846 mm^2.

Buckling class: Built up section. Hence it belongs to class 'c'.

Sectional Properties:

$$I_{zz} = 20458.4 \times 10^4 + 2 \times 300 \times 20 \times (200 + 10)^2$$

$$= 733784000 \text{ mm}^4$$

(*Note:* M.I. of plate about its own axis neglected)

$$I_{yy} = 622.1 \times 10^4 + \frac{1}{12} \times 20 \times 300^3 \times 2$$

$$= 96221000 \text{ mm}^4$$

$$\therefore \quad I_{yy} < I_{zz}$$

Buckling about y-y axis governs the design.

$$r = r_{yy} = \sqrt{\frac{I_{yy}}{A}}$$

$$A = 7846 + 2 \times 300 \times 20 = 19846 \text{ mm}^2$$

$$\therefore \quad r = r_{yy} = \sqrt{\frac{96221000}{19846}} = 69.63 \text{ mm}$$

Effective length $KL = 0.8\,L = 0.8 \times 4500 = 3600$ mm.

$$\therefore \quad \text{Slenderness ratio } \frac{KL}{r} = \frac{3600}{69.63} = 51.70$$

Referring to Table 6.4(c), for $\frac{KL}{r} = 51.70$

$$f_{cd} = 183 - \frac{1.70}{10}(183 - 168)$$

$$= 180.45 \text{ N/mm}^2$$

$$\therefore \quad P_d = A\,f_{cd} = 19846 \times 180.45$$

$$= 3581210 \text{ N}$$

$$= 3581.210 \text{ kN.}$$

$\therefore$ Load (working load) carrying capacity of the column $= \frac{3581.210}{1.5} = 2387.474$ kN **Answer.**

6.6 DESIGN OF COMPRESSION MEMBERS

The following are the usual steps in the design of compression members:

1. Design stress in compression is to be assumed.

   For rolled steel beam sections the slenderness ratio varies from 70 to 90. Hence design stress may be assumed as 135 N/mm^2. For angle struts, the slenderness ratio varies from 110 to 130. Hence design stress for such members may be assumed as 90 N/mm^2. For compression members carrying large loads, the slenderness ratio is comparatively small. For such members design stress may be assumed as 200 N/mm^2.
2. Effective sectional area required is $A = \frac{P_d}{f_{cd}}$.
3. Select a section to give effective area required and calculate r_{min}.
4. Knowing the end conditions and deciding the type of connection determine effective length.
5. Find the slenderness ratio and hence design stress f_{cd} and load carrying capacity P_d.
6. Revise the section if calculated P_d differs considerably from the design load.

Thus the design of compression member is by a trial and error process.

Example 6.4

Design a single angle strut connected to the gusset plate to carry 180 kN factored load. The length of the strut between centre to centre connection is 3 m.

Solution:

Assuming $f_{cd} = 90$ N/mm^2,

$$A = \frac{180 \times 10^3}{90} = 2000 \text{ mm}^2$$

Try ISA 9090, 12 mm, which has $A = 2019$ mm^2

$$r_{min} = r_{vv} = 17.4 \text{ mm.}$$

Assuming the strut will be connected to the gusset plate with at least 2 bolts (Note: Strength of 20 mm bolt in single shear is about 45 kN)

$$KL = 0.85L = 0.85 \times 3000 = 2550 \text{ mm}$$

$$\therefore \quad \frac{KL}{r} = \frac{2550}{17.4} = 146.55$$

From the Table 6.1(c),

for $f_y = 250$ N/mm^2

when $\frac{KL}{r} = 140, f_{cd} = 60.2$

when $\frac{KL}{r} = 150, f_{cd} = 59.2$

$\therefore$ when $\frac{KL}{r} = 146.55$

$$f_{cd} = 58.4 - \frac{6.55}{10}(58.4 - 52.6) = 54.6 \text{ N/mm}^2$$

$$\therefore \quad P_d = A f_{cd} = 2019 \times 54.6 = 110239 < 180000 \text{ N}$$

Hence revise the section.

Try ISA 130130, 8 mm.

Area provided = 2022 mm^2, $r = r_{vv} = 25.5$

$$\therefore \quad \frac{KL}{r} = \frac{2550}{25.5} = 100$$

$$\therefore \quad f_{cd} = 107 \text{ N/mm}^2$$

$$\therefore \quad P_{cd} = 2022 \times 107 = 216354 > 180{,}000 \text{ N.} \qquad \textbf{O.K.}$$

Provide ISA 130130, 8 mm.

Example 6.5

A column 4 m long has to support a factored load of 6000 kN. The column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam sections and plates.

Solution:

Assuming $f_{cd} = 200$ N/mm^2,

$$\text{Area required} = \frac{6000 \times 10^3}{200} = 30000 \text{ mm}^2$$

Using ISHB 450 @ 907 N/m,

Area provided = 11789 mm^2, width of flange = 250 mm.

$\therefore$ Area to be provided by plates = 30000 − 11789 = 18211 mm^2.

Selecting 20 mm plates, breadth required 'b' is obtained from,

$$2\, b \times 20 = 18211$$

$$b = 455.3$$

Provide 20 mm × 500 mm plate.

Check for overhang:

$$\text{Overhang } \frac{500 - 250}{20} = 12.5 < 12t \text{ (Clouse 10.2.3.2 in IS 800)} \qquad \textbf{O.K.}$$

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Total area provided

$$A_e = 11789 + 500 \times 20 \times 2$$

$$= 31789 \text{ mm}^2$$

For ISHB 450 @ 907 N/m

$$I_{zz} = 40349.9 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 3045 \times 10^4 \text{ mm}^4$$

$\therefore$ For the section selected,

$$I_{zz} = 40349.9 \times 10^4 + 2 \times 500 \times 20\,(225 + 10)^2$$

$$= 1507.994 \times 10^6 \text{ mm}^4$$

$$I_{yy} = 3045 \times 10^4 + 2 \times \frac{1}{12} \times 20 \times 500^3$$

$$= 447.1167 \times 10^6 \text{ mm}^4$$

$$\therefore \quad r = r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{447.1167 \times 10^6}{31789}}$$

$$= 118.6 \text{ mm}$$

Effective length $KL = 0.8\, L = 0.8 \times 4000 = 3200$ mm.

$$\therefore \quad \frac{KL}{r} = \frac{3200}{118.6} = 26.98$$

$$t_f = t_f \text{ of I section} + 20 = 13.7 + 20 = 31.7 < 40 \text{ mm.}$$

It belongs to buckling class c for buckling about y-y axis.

$\therefore$ From Table 6.1(c)

$$f_{cd} = 224 - \frac{6.98}{10}(224 - 211)$$

$$= 214.9 \text{ N/mm}^2$$

$$\therefore \quad P_d = A_e f_{cd} = 31789 \times 214.9$$

$$= 6831456 \text{ N}$$

$$= 6831.456 \text{ kN} > \text{Factored load}$$

Hence safe.

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6.7 LACED AND BATTENED COLUMNS

To achieve maximum value for minimum radius of gyration, without increasing the area of the section, a number of elements are placed away from the principal axis using suitable lateral systems. The commonly used lateral systems are

(a) lacing or latticing
(b) battening.

Perforated cover plates are also used for this purpose. However IS 800 do not give any specifications for the design of such plates.

6.7.1 Lacings

Rolled steel flats and angles are used for lacing. One can use single lacing or double lacing system (Fig. 6.5).

The object of providing lateral system is to keep the main members of the column away from principal ones. In doing so, the lacings are subjected to shear forces due to horizontal forces on columns.

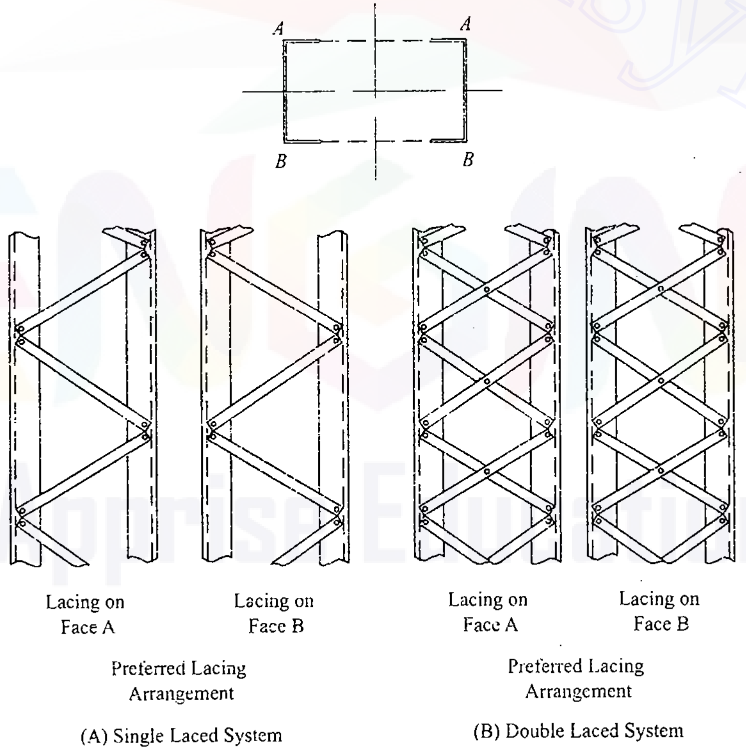

Figure 6.5 Laced column.

6.7.2 Battens

Instead of lacing one can use battens to keep members of columns at required distances. Figure 6.6 shows the use of batten plates.

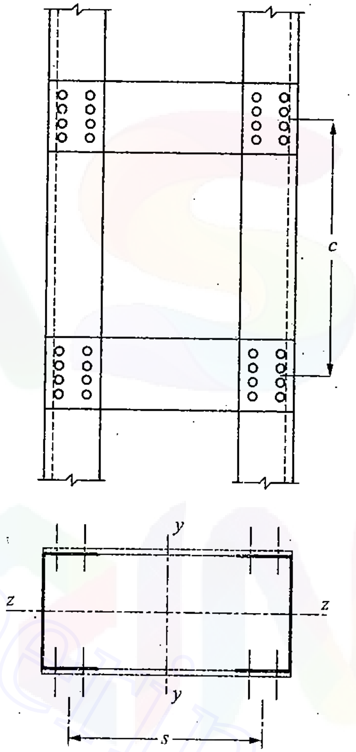

Figure 6.6 Battened column.

6.8 DESIGN OF LACED COLUMNS

IS 800-2007 specifies the following rules for the design of latticed columns:

1. As far as possible, the latticing system shall be uniform throughout.
2. In single laced system the direction of lattices on opposite faces should be shadow of the other. It should not be mutually opposite.

3. In bolted/riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the bolt/rivet.
4. The thickness of flat lacing bars shall not be less than $\frac{1}{40}$th of its effective length for single lacing and $\frac{1}{16}$th of the effective length for double lacings.
5. Lacing bars shall be inclined at 40° to 70° to the axis of built up member.
6. The distance between the two main members should be kept so as to get $r_{yy} > r_{zz}$ where, r_{yy} is the radius of gyration about weaker axis and r_{zz} is the radius of gyration of stronger axis of individual member (Fig. 6.5).
7. Maximum spacing of lacing bars shall be such that the maximum slenderness of the main member between consecutive lacing connection is not greater than 50 or 0.7 times the most unfourable slenderness ratio of the member as a whole.
8. The lacing shall be designed to resist transverse shear V_t = 2.5% of axial force in columns. If there are two transverse parallel systems then each system has to resist $\frac{V_t}{2}$ shear force.
9. If the column is subjected to bending also, V_t = bending shear + 2.5% column force.
10. Effective length of single laced system is equal to the length between the inner end fastener. For welded joints and double laced, effectively connected at intersection effective length may be taken as 0.7 times the actual length.
11. The slenderness ratio $\frac{KL}{r}$ for lacing bars should not exceed 145.
12. Laced compression members shall be provided with end tie plates.
13. The effective slenderness ratio of laced columns shall be taken as 1.05 times the actual maximum slenderness ratio, in order to account for shear deformation effects.

Example 6.6

Design a laced column with two channels back to back of length 10 m to carry an axial factored load of 1400 kN. The column may be assumed to have restrained in position but not in direction at both ends (hinged ends).

Solution:

Assuming $f_{cd} = 135$ N/mm^2

$$\text{Area required} = \frac{1400 \times 1000}{135} = 10370 \text{ mm}^2$$

Try 2 ISMC 350 @ 413 N/m.

Area provided = 2 × 5366 = 10732 mm^2

$r_{zz} = 136.6$ mm

Distance will be maintained so as to get $r_{yy} > r_{zz}$.

$$\therefore \quad \text{Actual } \frac{KL}{r} = \frac{1 \times 10000}{136.6} = 73.206$$

Since it is a laced column

$$\frac{KL}{r} = 1.05 \times 73.206 = 76.87$$

From Table 6.1(c),

$$f_{cd} = 152 - \frac{6.87}{10}(152 - 136)$$

$$= 141.0 \text{ N/mm}^2$$

Load carrying capacity = 10732 × 141.0

= 1513.297 × 10^3

= 1513.297 kN > 1400 kN **O.K.**

Spacing between the channels:

Let it be a clear distance '*d*',

Now: $I_{xx} = 2 \times 10008 \times 10^4 = 20016 \times 10^4$ mm^4

$$I_{yy} = 2\left[430.6 \times 10^4 + 5366\left(\frac{d}{2} + 24.4\right)^2\right]$$

Equating I_{yy} to I_{xx}, we get

$$2\left[430.6 \times 10^4 + 5366\left(\frac{d}{2} + 24.4\right)^2\right] = 20016 \times 10^4$$

$$\text{i.e.,} \quad \left(\frac{d}{2} + 24.4\right)^2 = 17848.3$$

$$\therefore \quad d = 218.4 \text{ mm}$$

Provide d = 220 mm as shown in Fig. 6.7.

Lacings: Let the lacings be provided at 45° to horizontal.

Horizontal spacing of lacing = 220 + 60 + 60

= 340 mm [Note: g = 60 is gauge distance]

∴ Vertical spacing = 340 tan 45° × 2

= 680 mm

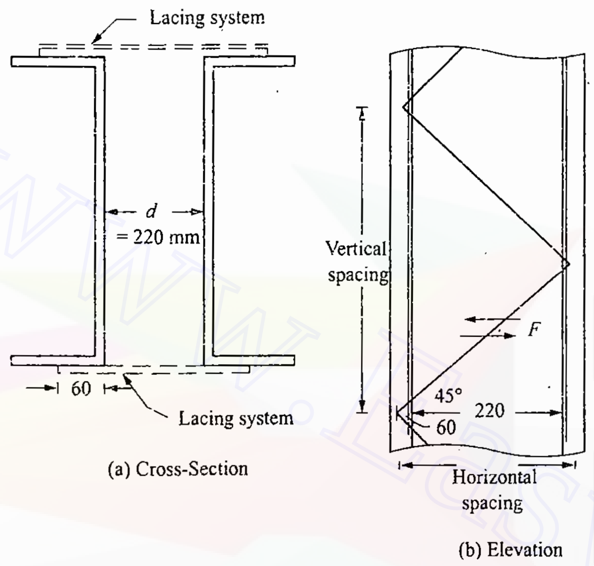

Figure 6.7

Least r of one channel $= r_{yy} = 28.3$

$\therefore \quad \frac{KL}{r}$ of channel between lacing $= \frac{680}{28.3} = 24.03 < 50$ **O.K.**

Transverse shear to be resisted by lacing systems $= \frac{2.5}{100} \times 1400 \times 10^3 = 35000$ N.

Shear to be resisted by each lacing systems $= \frac{35000}{2} = 17500$ N.

Length of lacing $= (220 + 60 + 60)\ \frac{1}{\cos 45} = 480.83$ mm.

Minimum thickness of lacing $= \frac{1}{40} \times 480.83$

$= 12.02$ mm.

Use 14 mm flats

Minimum width of lacing, if 20 mm bolts are used $= 3 \times 20 = 60$ mm.

Use 60 ISF 14

Sectional area $= 60 \times 14 = 840$ mm^2.

$$r_{min} = \sqrt{\frac{\frac{1}{12} \times 60 \times 14^3}{60 \times 14}} = 4.041 \text{ mm}$$

$$\therefore \quad \frac{KL}{r} = \frac{480.83}{4.041} = 118.97 < 145$$ **O.K.**

Strength of 20 mm shop bolt:

(a) in single shear $= 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272$ N

Edge distance $= \frac{60}{2} = 30$

$$\therefore \quad K_b = \frac{30}{3 \times 22} = 0.4545$$

(b) Strength in bearing $= \frac{2.5 K_b\, d\, t\, f_u}{1.25}$

$$= \frac{2.5 \times 0.4545 \times 20 \times 10 \times 400}{1.25}$$

$= 101808$ N

$\therefore$ Bolt value $= 45272$ N

Number of bolt required $= \frac{17500}{45272} = 0.387$

Provide one bolt.

Check for the strength of lacing:

$$\frac{KL}{r} = 118.97$$

A flat belongs to bucking class c.

$$\therefore \quad f_{cd} = 94.6 - \frac{8.97}{10}\ (94.6 - 83.7)$$

$= 84.82$ N/mm^2

Load carrying capacity in compression $= 84.82 \times 60 \times 14 = 71251$ N

Force in lacing $= \dfrac{17500}{\sin 45} = 24749 \text{ N} < 71251 \text{ N}$

$\therefore$ Safe.

Hence provide 60 ISF 14 flats at 45° and connect them to centre of gravity of channels with one bolt of 20 mm nominal diameter.

6.9 DESIGN OF BATTENED COLUMNS

IS 800-2007 specifies the following rules for the design of battened columns:

1. Batten plates should be provided symmetrically.
2. At both ends batten plates should be provided. They should be provided at points where the member is stayed in its length.
3. The number of battens should be such that the member is divided into not less than three bays. As far as possible they should be spaced and proportioned uniformly throughout.
4. Battens shall be of plates, angles, channels, or I-sections and at their ends shall be riveted, bolted or welded.
5. By providing battens distance between the members of columns is so maintained that radius of gyration about the axis perpendicular to the plane of battens is not less than the radius of gyration about the axis parallel to the plane of the batten ($r_{yy} > r_{xx}$, in Fig. 6.6).
6. The effective slenderness ratio of battened columns shall be taken as 1.1 times the maximum actual slenderness ratio of the column, to account for shear deformation.
7. The vertical spacing of battens, measured as centre to centre of its end fastening, shall be such that the slenderness ratio of any component of column over that distance shal! be neither greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its *z-z* axis.
8. Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force V_t equal to 2.5% of the total axial force.
9. In case columns are subjected to moments also, the resulting shear force should be found and then the design shear is sum of this shear and 2.5% of axial load.
10. The design shear and moments for batten plates is given by

$$V_b = \frac{V_t C}{NS} \quad \text{and} \quad M = \frac{V_t C}{2N} \quad \text{at each connection.}$$

where,

V_t = transverse shear force as defined in 8 and 9.

C = distance between centre to centre of battens longitudinally.

N = number of parallel planes.

S = minimum transverse distance between the centroid of the fasteners connecting batten to the main member.

11. The effective depth of end battens (longitudinally), shall not be less than the distance between the centroids of main members.
12. Effective depth of intermediate battens shall not be less than ¾th of above distance.
13. In no case the width of battens shall be less than twice the width of one member in the plane of the batten. It is to be noted that the effective depth of a batten shall be taken as the longitudinal distance between the outermost fastners.
14. The thickness of battens shall be not less than $\frac{1}{50}$th of the distance between the innermost connecting lines of rivets, bolts or welds.
15. The length of the weld connecting batten plate to the member shall not be less than half the depth of batten plate. At least one third of the weld shall be placed at each end of this edge.

Example 6.7

Design the built up column of example 6.6 using battens instead of lacing system.

Solution:

The design of column is same as in the previous example i.e., use 2ISMC 350 @ 413 N/m with clear spacing of 220 mm. $\dfrac{KL}{r} = 1.1 \times \dfrac{10000}{136.6} = 80.52$

Distance between centres of channels $S = 220 + 60 + 60 = 340$ mm

Design of battens:

Let C be the spacing of battens, longitudinally.

Radius of gyration of one channel = 28.3 mm

$$\therefore \quad \frac{C}{28.3} < 50 \quad \text{i.e.,} \quad C < 1415.$$

It should also satisfy the condition,

$$\therefore \quad \frac{C}{28.3} < 0.7 \times 80.52 \quad \text{i.e.,}\ C < 1595.$$

Let us select $C = 1200$ mm.

$$V_t = \frac{2.5}{100} \times 1400 \times 10^3 = 35000 \text{ N}$$

$$\therefore \quad V_b = \frac{V_t C}{NS} = \frac{35000 \times 1200}{2 \times 340} = 61765 \text{ N}$$

$$M = \frac{V_t C}{2N} = \frac{35000 \times 1200}{2 \times 2} = 10500000 \text{ N-mm}$$

Size of battens:

Effective depth of end batten $\not< 268.8$ mm and also $\not< 2 \times 100$ mm.

$\therefore$ Provide 270 mm depth for end battens, overall depth $= 270 + 2 \times 35 = 340$ mm.

For intermediate battens it is $\not< \frac{3}{4} \times 270$ mm and $\not< 200$ mm

Provide depth = 210 mm

Giving edge distance of 35 mm,

Overall depth $= 210 + 2 \times 35 = 280$ mm

Thickness of battens $\not< \frac{1}{50} \times 340$

$\not< 6.8$

Use 8 mm thick plates.

Check for stresses in batten plates:

$$\text{Shear stress} = \frac{61765}{280 \times 8} = 27.57 \text{ N/mm}^2 < \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \qquad \text{O.K.}$$

Shear stress < 0.6 × permissible stress

$$\text{Bending stress} = \frac{6M}{td^2} = \frac{6 \times 10500000}{8 \times 280^2} < \frac{f_y}{1.1} \times 1.2$$

$$= 100.45 < 227.27 \text{ N/mm}^2 \qquad \text{O.K.}$$

Obviously end plate satisfies these requirements since it is deeper.

Connections:

It is to be designed to transmit both shear and bending moment.

Using 20 mm bolts,

$$\text{Strength in single shear} = 0.78 \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272 \text{ N}$$

Strength in bearing is much higher.

$\therefore$ Bolt value = 45272 N.

$$\text{Number of bolts required} = \frac{78125}{45272} = 1.72$$

Let us provide 3 bolts to take into account stresses due to bending also.

Check:

$$\text{Force in each bolt due to shear} = \frac{78125}{3} = 26042 \text{ N}$$

Let the pitch be $\frac{210}{2} = 105$ mm

$$\text{Force due to moment in extreme bolt} = \frac{Mr}{\Sigma r^2}$$

$$= \frac{10500000 \times 105}{105^2 + 105^2} = 50000$$

$$\text{Resultant force in extreme bolt} = \sqrt{26042^2 + 50000^2} > 45272 \text{ N}$$

Try 5 bolts as shown in Fig. 6.8.

$$\text{i.e. Force in each bolt due to shear} = \frac{78125}{5} = 15625 \text{ N}$$

$$\text{Force due to moment in extreme bolt} = \frac{Mr}{\Sigma r^2} = \frac{10500000 \times 100}{2\left(50^2 + 100^2\right)} = 42000 \text{ N}$$

$$\therefore \text{Resultant force} = \sqrt{15625^2 + 42000^2}$$

$$= 44812 \text{ N} < 45272 \text{ N}$$

Provide the bolts as shown in Fig. 6.8.

6.10 COLUMN SPLICE

Connecting two pieces of sections to get the required length of column is called column splicing. In multistorey buildings, the section of the column may be changed from storey to storey for economy. This also creates the need for splicing. In such cases column is preferably spliced at the point of inflection, which is usually 150 to 300 mm above the floor line. There are two distinct types of compression splices:

(i) those having ends cut by ordinary method.
(ii) those having the ends cut and milled.

If the ends are not milled, the splice plates and their connections to the column are designed to transmit all forces. The columns having milled ends, the ends are placed firmly in contact with each other and hence considerable load is transferred by bearing. The connections and splice plates are designed for only 50 percent of axial load.

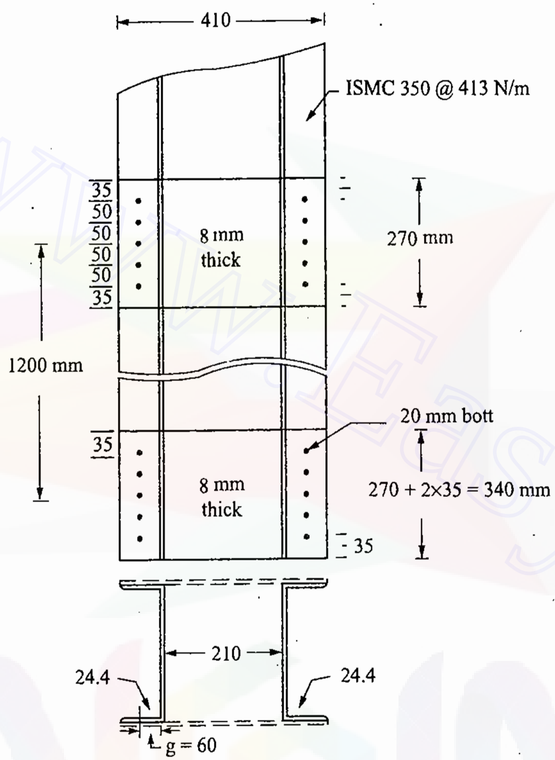

Figure 6.8

The various types of column splices used are shown in Fig. 6.9. The situations in which they are used are:

(a) When the columns are of the same size, milled ends are provided.
(b) When columns are of slightly different sizes, filler plates are used. Load is transferred partially by bearing.
(c) When the columns are of considerably different sizes, bearing plates are used.

6.11 DESIGN OF COLUMN SPLICES

The following procedure may be used in the design of column splices.

(1) Column splice plates may be assumed to act as short columns of zero slenderness ratio i.e. assume $f_{cd} = \frac{f_y}{1.1}$ and calculate required area.

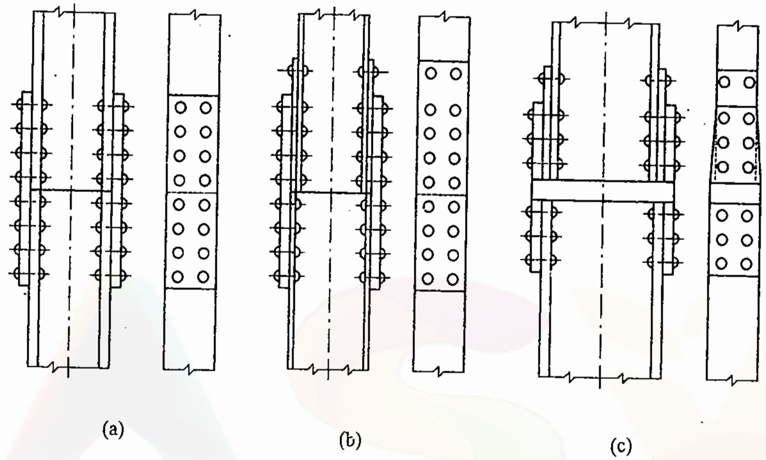

Figure 6.9 Column splices.

(2) Width of splice plate is taken equal to that of flange of column and the required thickness calculated.
(3) For the selected diameter of the bolts, the bolt value is computed and the number required is found.
(4) If the moment and shear force are also acting in addition to axial load splice plates are provided to flanges as well as to web. Splice plates attached to flanges are designed to resist additional axial load equal to $\frac{M}{a}$, where a is the distance between centre to centre of flange splice plates. The web splice plates are designed to resist maximum shear force.
(5) When bearing plates are to be provided to join two columns of unequal sizes the following steps are to be used for the design of bearing plates.

(i) Bearing plate may be assumed as short beam to transmit the axial load to the lower column.
(ii) Axial load of the column is assumed to be taken by flanges only. Thus the load transfer is as shown in Fig. 6.10.

Hence maximum moment in bearing plate $= \frac{P}{2}a$

$\therefore$ The thickness 't' of bearing plate required is given by $\frac{1}{6}bt^2 f_{bs} = M$ or $t = \sqrt{\frac{6M}{bf_{bs}}}$

where f_{bs} = design bending stress = $\frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27 \text{ N/mm}^2$

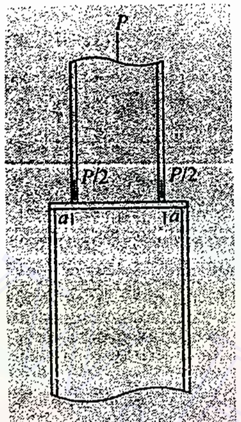

Figure 6.10

Example 6.8

A column section ISHB 300 @ 577 N/m is carrying a factored axial load of 600 kN, a factored moment of 30 kN–m and a factored shear force of 60 kN. Design a suitable column splice. Assume ends are milled.

Solution:

Since the ends are milled, 50% of axial load is transferred through bearing and splice plates transfer the remaining 50% of the load.

$\therefore$ Load to be transferred by splice plate = 300 kN

i.e., load to be transferred by each splice plate = 150 kN

Assuming the thickness of splice plate 6 mm, for the calculation of lever arm,

$a = 300 + 6 = 306$ mm.

$$\therefore \text{ Force in each plate due to moment } = \frac{30 \times 10^3}{306} = 98.04 \text{ kN}$$

$$\therefore \text{ Total load in each splice plate} = 150 + 98.04 = 248.04 \text{ kN}$$

For rolled steel section, $f_y = 250$ N/mm^2.

$$\text{Area required } = \frac{248.04 \times 10^3}{250/1.1} = 1091.376 \text{ mm}^2$$

Width of splice plate = width of flange = 250 mm.

$$\therefore \text{ Thickness required } = \frac{1091.376}{250} = 4.365 \text{ mm}$$

Provide 6 mm plates.

Using 20 mm bolts of grade 4.6,

$$\text{Strength in single shear } = 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272 \text{ N}$$

Strength in bearing is much higher, if more than minimum specified edge distance is provided.

$\therefore$ Bolt value = 45272 N

$$\text{Number of bolts required } = \frac{248.04 \times 10^3}{45272} = 5.47$$

Provide 3 bolts on each side of web.

To resist shear, splice plates are provided on each side of web.

Maximum shear force = 60 kN.

$$\text{Yield strength of the web [clause 8.4.1 in IS 800]} = \frac{f_{yw}}{\sqrt{3}} = \frac{250}{\sqrt{3}} = 144.3 \text{ N/mm}^2$$

$$\text{Area of plates required } = \frac{60 \times 10^3}{144.3} = 416 \text{ mm}^2$$

$\therefore$ Area of each plate = 208 mm^2. Use 6 mm plates as shown in Fig. 6.11.

$$\text{Number of bolts required } = \frac{60 \times 10^3}{45272} = 1.33$$

Provide 2 bolts. Using 6 mm web splice the arrangement may be made as shown in Fig. 6.11.

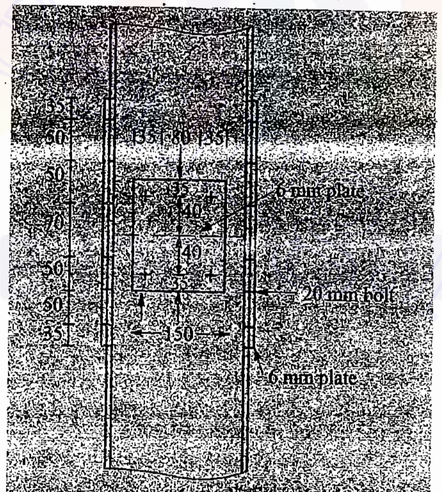

Figure 6.11

Example 6.9

An upper storey column ISHB 300 @ 577 N/m carries a factored load of 1200 kN and a factored moment of 12 kN-m. It is to be spliced with lower storey column ISHB 400 @ 806 N/m. Design a suitable splice.

Solution:

Assuming that column load is transferred by flanges only,

Load on each flange $= \frac{1200}{2} = 600$ kN.

Distance between the flanges of ISHB 300 @ 577 N/m = 300 – 10.6 = 289.4 mm

∴ Distance between the flanges in ISHB 400 @ 806 N/m = 400 – 12.7 = 387.3 mm.

∴ Distance $a = \frac{387.3 - 289.4}{2}$

= 48.95 mm [Ref. Fig. 6.12]

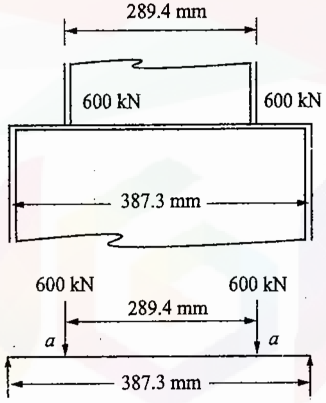

Figure 6.12

∴ Moment on bearing plate = 600 × 48.95 kN-mm

Width of bearing plate = Width of flange

= 250 mm

Design bending stress $= \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27$ N/mm^2

Equating moment of resistance to bending moment, we get,

$$\frac{1}{6} bt^2 f_{bs} = M$$

$$\frac{1}{6} \times 250\, t^2 \times 227.27 = 600 \times 48.95 \times 10^3$$

∴ $t = 55.69$ mm

Adopt 56 mm thickness.

Splice Plate:

Column ends are milled for complete bearing. Hence splice plates are designed for 50% of load

Load on splice plates $= \frac{1200}{2} = 600$ kN.

∴ Load on each splice plate = 300 kN.

Assuming 6 mm thick plates, distance between splice plates = 400 + 6 = 406 mm.

∴ Load due to bending $= \frac{12 \times 10^3}{406} = 29.6$ kN

Hence, Total load = 300 + 29.6 = 329.6 kN

Width of splice plate = Width of column

= 250 mm.

∴ Thickness of splice plate $= \frac{329.6 \times 1000}{250 \times 227.27} = 5.8$ mm

Provide 6 mm plates.

Bolts: Using 20 mm bolts

Strength in single shear $= 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 45272$ N

Bearing strength is higher as long as minimum end distance is maintained.

∴ Bolt value = 45272 N.

No. of bolts required $= \frac{329.4 \times 1000}{45272} = 7.3$

Provide 8 bolts in two rows on each plate as shown in Fig. 6.13.

Thickness of filler plate $= \frac{400 - 300}{2} = 50$ mm > 6 mm.

∴ According to clause 10.3.3.3,

$$\beta_{pk} = 1 - 0.0125\, t_{pk}$$

$$= 1 - 0.0125 \times 50 = 0.375$$

$\therefore$ Shear capacity of bolt = (0.375) 45272

= 16977 N

$\therefore$ Number of bolts required to connect splice plate with ISHB 300 @ 577 N/m

$$= \frac{329.4 \times 1000}{16977} = 19.4$$

$\therefore$ Provide 12 additional bolts, 6 on each side in the filler portion.

The details of connection are shown in Fig. 6.13.

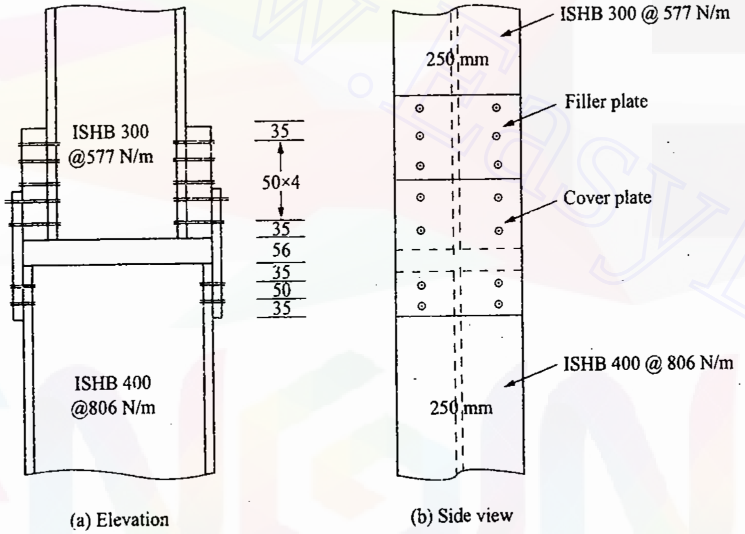

Figure 6.13

6.12 COLUMN BASES

Column bases transmit the column load to the concrete or masonry foundation blocks. The column base spreads the load on wider area so that the intensity of bearing pressure on the foundation block is within the bearing strength. There are two types of column bases commonly used in practice:

1. Slab Base
2. Gusseted Base.

6.12.1 Slab Base

These are used in columns carrying small loads. In this type, the column is directly connected to the base plate through cleat angles as shown in Fig. 6.14. The load is transferred to the base plate through bearing.

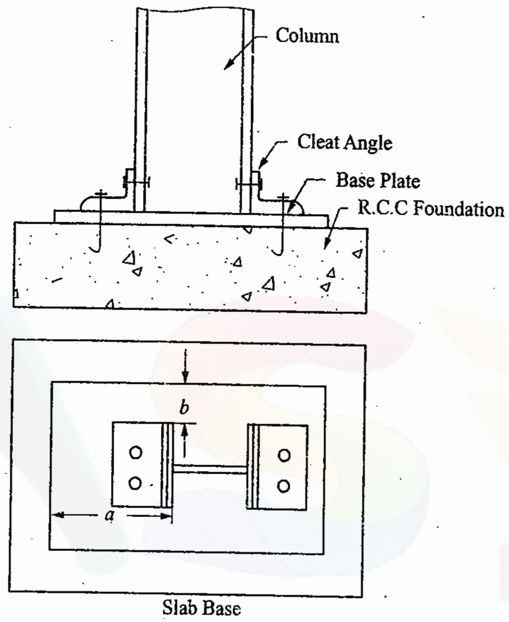

Figure 6.14 Slab base.

6.12.2 Gusseted Base

For columns carrying heavy loads gusseted bases are used. In gusseted base, the column is connected to base plate through gussets. The load is transferred to the base partly through bearing and partly through gussets. Figure 6.15 shows a typical gusseted base connection.

6.13 DESIGN OF SLAB BASE

The design of slab base consists in finding the size and thickness of slab base. In the procedure given below it is assumed that the pressure is distributed uniformly under the slab base.

Size of Base plate:

(1) Find the bearing strength of concrete which is given by = $0.45 f_{ck}$

(2) Therefore, area of base plate required $= \dfrac{P_u}{0.45 f_{ck}}$, where P_u is factored load.

(3) Select the size of base plate. For economy, as far as possible keep the projections a and b equal.

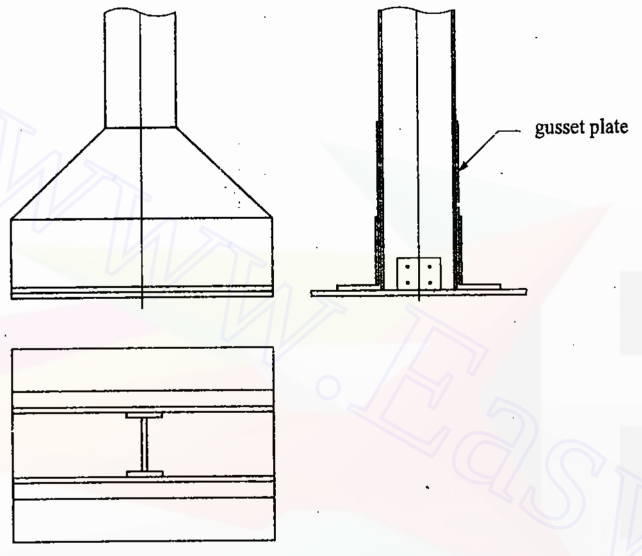

Figure 6.15

Thickness of Base Plate:

(1) Find the intensity of pressure

$$w = \frac{P_u}{\text{Area of base plate}}$$

(2) Minimum thickness required is given by

$$t_s = \left[\frac{2.5w\left(a^2 - 0.36^2\right)\gamma_{mo}}{f_y}\right]^{0.5} > t_f$$

where

t_s = thickness of base plate

and t_f = thickness of flange.

The above formula may be derived by taking $\mu = 0.3$ and using plate theory for finding bending moment.

Connections:

(1) Connect base plate to foundation concrete using four 20 mm diameter and 300 mm long anchor bolts.

(2) If bolted connection is to be used for connecting column to base plate, use 2 ISA 6565, 6 mm thick angles with 20 mm bolts.

(3) If weld is to be used for connecting column to base check the weld length of fillet welds.

Example 6.10

Design a slab base for a column ISHB 300 @ 577 N/m carrying an axial factored load of 1000 kN. M20 concrete is used for the foundation. Provide welded connection between column and base plate.

Solution:

Bearing strength of concrete $= 0.45\, f_{ck}$

$$= 0.45 \times 20 = 9 \text{ N/mm}^2$$

Factored load $P_u = 1000$ kN.

$$\therefore \quad \text{Area of base plate required} = \frac{1000 \times 10^3}{9}$$

$$= 111111 \text{ mm}^2$$

Provide 360 × 310 size plate.

Area provided = 360 × 310 = 111600 mm^2.

$$\text{Pressure} = \frac{1000 \times 10^3}{111600} = 8.96 \text{ N/mm}^2$$

Projections are

$$a = \frac{360 - 300}{2} = 30 \text{ mm}$$

$$b = \frac{310 - 250}{2} = 30 \text{ mm}$$

$$\therefore \quad t_s = \left[\frac{2.5 \times 8.96\left(30^2 - 0.3 \times 30^2\right) \times 1.1}{250}\right]^{0.5}$$

$$= 7.88 \text{ mm.}$$

Thickness of flange of ISHB 300 @ 577 N/m is 10.6 mm.

Provide 12 mm thick plate.

Connecting 360 × 310 × 12 mm plate to concrete foundation:

Use 4 bolts of 20 mm diameter 300 mm long to anchor the plate.

Welds: Properly machined column is to be connected to base plate using fillet weld.

Total length available for welding (Ref. Fig. 6.16)

$$= 2(250 + 250 - 7.6 + 300 - 2 \times 10.6) = 1542.4 \text{ mm.}$$

$$\text{Strength of weld} = \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 189.37 \text{ N/mm}^2$$

Let 's' be the size of weld. Then effective area of weld = 0.7 s L_e

where L_e is effective length.

$\therefore$ The design condition is $0.7\, s\, L_e \times 189.37 = 1000 \times 10^3$

$s L_e = 7543.8$

Using 6 mm weld, L_e = 1257 mm.

After deducting for end return of the weld at the rate of twice the size of the weld at each end.

Available effective length = 1542.4 – 2 × 6 × No. of returns (corners)

= 1542.4 – 2 × 6 × 12

= 1398.4 > 1257 mm.

Hence 6 mm weld is adequate.

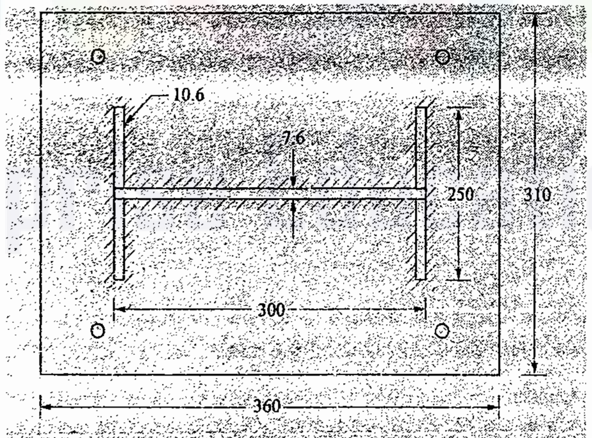

Figure 6.16

6.14 DESIGN OF GUSSETED BASE

IS 800-2007 specifies that the gusset plates, angle cleats, stiffeners and fastenings etc., in combination with the bearing area, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact.

The following design procedure may be followed:

1. Area of base plate $= \dfrac{\text{Factored Load}}{0.45\, f_{ck}}$
2. Assume various members of gusset base.
   (a) Thickness of gusset plate is assumed as 16 mm.
   (b) Size of the gusset angle is assumed such that its vertical leg can accommodate two bolts in one vertical line. Corresponding to this leg the other leg is assumed in which one bolt can be provided.
   (c) Thickness of angle is kept approximately equal to the thickness of gusset plate.
3. Width of gusset base is kept such that it will just project outside the gusset angle and hence length $= \dfrac{\text{Area of plate}}{\text{width}}$.
4. When the end of the column is machined for complete bearing on the base plate, 50 percent of the load is assumed to be transferred by the bearing and 50 percent by the fastenings.
   When the ends of the column shaft and gusset plates are not faced for complete bearing, the fastenings connecting them to the base plate shall be designed to transmit all the forces to which the base is subjected.
5. The thickness of the base plate is computed by flexural strength at the critical sections.

Example 6.11

Design a gusseted base for a column ISHB 350 @ 710 N/m with two plates 450 mm × 20 mm carrying a factored load of 3600 kN. The column is to be supported on concrete pedastal to be built with M20 concrete.

Solution:

$f_{ck} = 20 \text{ N/mm}^2$

$$A = \frac{P_u}{0.45 f_{ck}} = \frac{3600 \times 10^3}{0.45 \times 20} = 400000 \text{ mm}^2$$

Selecting ISA 150 115, 15 mm angle and 16 mm thick gusset plate (Fig. 6.17).

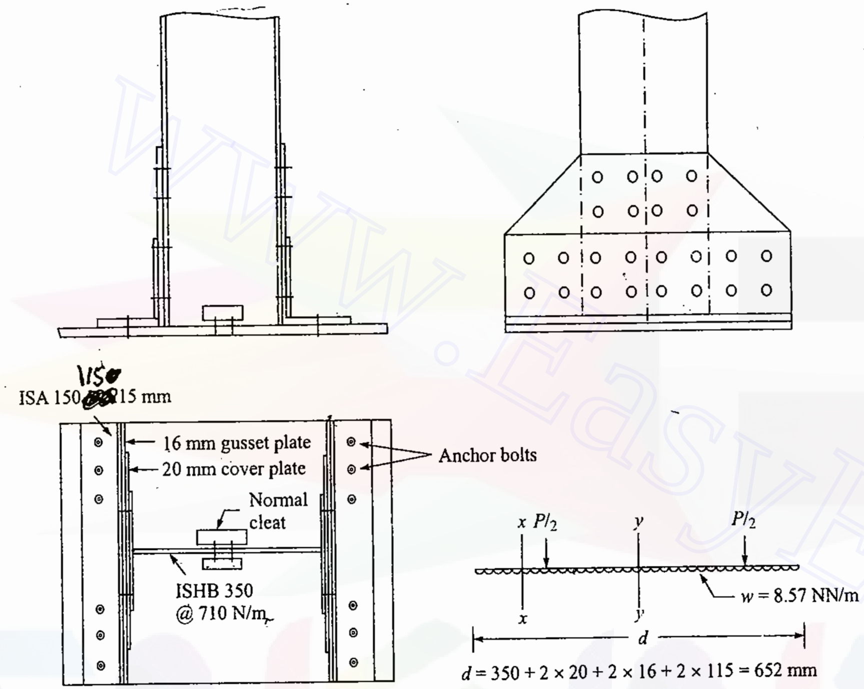

Figure 6.17

Minimum width required $= 350 + 2 \times 20 + 2 \times 16 + 2 \times 115$

$= 652$ mm.

Use 700 mm wide plate.

$\therefore$ Length of base plate $= \dfrac{400000}{700} = 571$ mm

Provide 700×600 mm plate.

Pressure under base plate $= \dfrac{3600 \times 10^3}{700 \times 600} = 8.57$ N/mm^2

$$a = \frac{700 - (350 + 20 \times 2 + 16 \times 2 + 2 \times 15)}{2} = 124 \text{ mm}$$

BM at section X-X per mm width

$$= 8.57 \times \frac{124^2}{2} = 65886 \text{ N-mm}$$

At section Y-Y, bending moment [Note: Per mm width $P = 8.57 \times 350$ N]

$$M_{yy} = 8.57 \times \frac{350^2}{2} - \frac{700}{2} \times 8.57 \times \left(\frac{350}{2} + 20 + \frac{16+15}{2}\right)$$

$$= 106482 \text{ N-mm}$$

$\therefore$ Design moment $= 106482$ N-mm.

Bending strength $= \dfrac{f_y}{\gamma_{mo}} = \dfrac{250}{1.1} = 227.27$ N/mm^2

Equating moment of resistance to bending moment we get,

$$1.2 \times \frac{1}{6} \times 1 \times t^2 \times 227.27 = 106482$$

$\therefore$ $t = 48.4$ mm.

$\therefore$ Use 56 mm base plate of size 700×600 mm.

Assuming ends of columns are faced for complete bearing, the connection between gusset plate and column will be designed for 50 percent of axial load.

Design load $= 0.5 \times 3600 = 1800$ kN.

Load on each splice $= \dfrac{1800}{2} = 900$ kN.

Using 24 mm shop bolts,

Strength of bolt in single shear $= 0.78 \times \dfrac{\pi}{4} \times 24^2 \times \dfrac{400}{\sqrt{3}} \times \dfrac{1}{1.25}$

$= 65192$ N.

Strength in bearing is higher.

$\therefore$ Bolt value $= 65192$ N.

$\therefore$ No. of bolts required $= \dfrac{900 \times 10^3}{65192} = 13.8$

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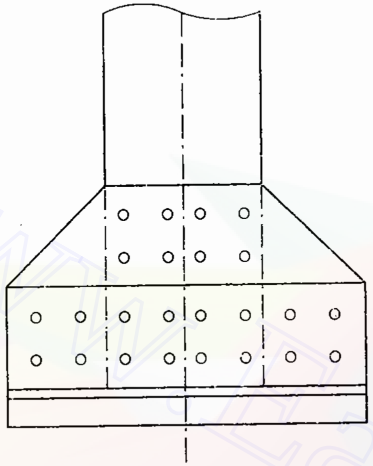

Figure 6.18

Provide 16 bolts as shown in Fig. 6.18, for connecting column to gusset plate. Use another 8 bolts to connect cleat angle to gusset plate.

* * *

Questions

1. With neat sketches explain different types of the following:
   (a) splices
   (b) base connections.
2. Determine the load carrying capacity of a strut made with ISA 10075, 10 mm, if its length is 2.8 m in the following cases of end connections:
   (a) one bolt used
   (b) two bolts used
   (c) welded rigidly to gusset plate.
3. Determine the load carrying capacity of a strut made with 2 ISA 7575, 6 mm, back to back if the length of member is 3.0 m and welded to a 12 mm gusset plate.
4. An ISMB 150 is used as a column. It is laterally supported in the plane of the major axis at a height 2.5 m and in the plane of minor axis at a height of 4.5 m. The ends may be assumed as hinged. What will be the allowable load on the column?

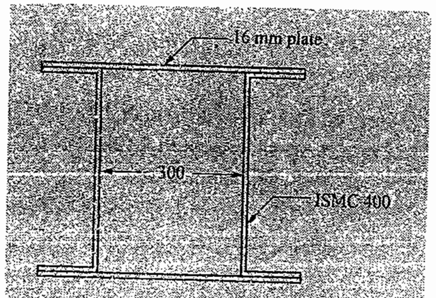

Figure 6.19

5. Determine the allowable compressive load which the member shown in Fig. 6.19 can support if the member is having 5.5 m effective length. Assume E 250 (Fe 415) grade steel.
6. Design a double angle strut to carry an axial factored load of 240 kN. The length of strut is 3.0 m. Bolted connections are to be used to connect it to 12 mm gusset plate.
7. A column 5 m long, has to support a factored load of 3600 kN. The column is held effectively at both ends and restrained in direction at one end. Design the column using beam sections and plates.
8. A column of 9 m effective length has to support an axial factored load of 1500 kN. Design the column which shall consist of two channels placed back to back at suitable spacing. Design also single angle lacing system.
9. Design a built up column consisting of two channels placed toe to toe. The column carries an axial factored load of 1600 kN. The effective height of the column is 10 m. Design the lacing also.
10. Design the column given in example 6.9 using battens instead of lacing system.
11. A column section ISHB 350 @ 710 N/m is carrying a factored load of 800 kN, a factored moment of 30 kN-m and a factored shear of 80 kN. Assuming ends are milled, design a suitable column splice.
12. An upper storey column ISHB 300 @ 577 N/m carries a factored load of 1200 kN and a factored moment of 12 kN-m. It is to be spliced with lower storey column ISHB 350 @ 710 N/m. Design a suitable splice.
13. A steel column ISHB 250 @ 537 N/m supports a total factored load of 1000 kN. Design a slab base for the column. The column is supported on a pedestal made of M20 concrete.
14. Design a gusseted base to carry an axial factored load of 3000 kN. The column is ISHB 450 @ 855 N/m with two 250×22 mm cover plates on either side. The effective height of the column is 5 m. The column is to rest on M20 concrete pedestal.

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13. A steel column ISHB 250 @ 537 N/m supports a total factored load of 1000 kN. Design a slab base for the column. The column is supported on a pedestal made of M20 concrete.
14. Design a gusseted base to carry an axial factored load of 3000 kN. The column is ISHB 450 @ 855 N/m with two 250 × 22 mm cover plates on either side. The effective height of the column is 5 m. The column is to rest on M20 concrete pedestal.

7

DESIGN OF BEAMS

Beam is a structural member with length considerably larger than cross sectional dimensions subject to lateral loads which give rise to bending moment shear forces in the member. Purlins which rest between the trusses and support roof sheets are beams. For this, angles or channels are commonly used. T-sections are used in water tanks to support steel plates. In buildings, I-sections are commonly used as beams. For heavier loads I-sections with additional plates connected on flanges are used. If still heavier sections are required built up sections like plate girders are used.

Based on the lateral supports to compression flanges there are mainly two types of beams viz., (a) Laterally supported beams and (b) Laterally unsupported beams.

If the compression flanges are laterally supported by flooring, it is mainly subjected to bending and shear. If the compression flange of beam is not laterally supported, the lateral buckling of the compression flange reduces the load carrying capacity of the beam.

In this chapter the design of both type of beams is presented based on limit state consideration as recommended by IS 800-2007.

7.1 PLASTIC MOMENT CARRYING CAPACITY OF A SECTION

Consider the cross section of a simply supported beam where the bending moment is maximum for the given loading. Within the elastic limit the stress varies linearly from compression to tension as shown in Fig. 7.1(b). As the load is gradually increased stresses increase proportionately till extreme fibre is subjected to yield stress. Then extreme fibre yields (Fig. 7.1(c)). For simplicity of analysis, stress strain for steel is assumed as shown in Fig. 7.2, in which strain hardening part of the curve is ignored and it is assumed that after yield point is reached fibre goes on yielding without resisting any additional load. Hence according to theory of plastic analysis highly stressed fibre once yields is not capable of resisting any moment. But interior fibres are not yet yielded and hence additional loads are resisted by unyielded portion of the section. As the load is gradually increased one by one fibre reach yield stress and stop resisting additional load. Figure 7.1(d) shows partially yielded case. However resistance to load continues till all fibres are yielded as shown in Fig. 7.1(e). After this condition the section will not resist further

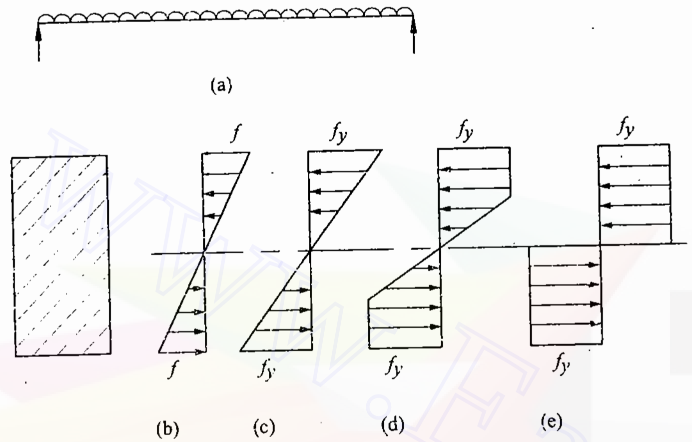

Figure 7.1

moment due to increase in load. This condition when all fibres at a section yield is called formation of plastic hinge. After this stage the rotation at section will take place without resisting additional moment but the moment corresponding to yielding of all fibres is resisted. This moment capacity is called plastic moment capacity of the section and is denoted as M_p.

The expression of M_p of a section can be easily derived as given below:

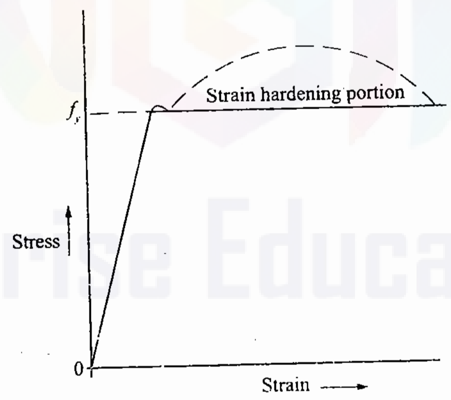

Figure 7.2

Let the area of the section in compression be A_c, in tension be A_t and total area A. Equating the horizontal forces for the equilibrium condition, we get

$$A_c f_y = A_t f_y$$

$$\therefore \quad A_c = A_t = \frac{A}{2}$$

Denoting the section where stress changes the sign as plastic neutral axis, we can conclude plastic neutral axis divides the total area into two equal parts. Obviously such section is at mid depth for symmetric sections as shown in Figs. 7.3(a and b). For unsymmetric sections it is to be found from the condition that $A_c = A_t = \frac{A}{2}$. This is shown in Fig. 7.3(c).

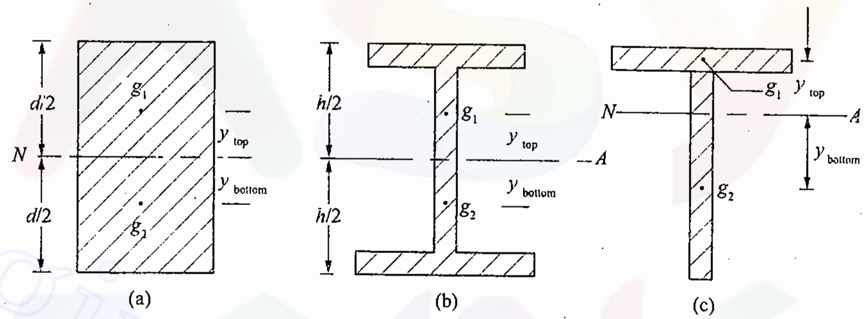

Figure 7.3 Plastic neutral axis.

Plastic moment capacity may be found by taking moment of horizontal forces about plastic N-A. It may be noted that the total moment of resistance is additive of moment resisted by compressive forces and tensile forces, since the moments are having same sign (clockwise or anticlockwise). Three examples are solved to illustrate the method of finding M_p and hence Z_p (Ref. Ex 7.1) [$M_p = f_y Z_p$].

For standard rolled sections it is found that Z_p is of the order of 1.125 to 1.14 times Z_{xx} for I sections and about 1.7 to 1.8 for channel sections. If these rolled sections are treated as the sections with rectangular parts, it is possible to determine Z_p values but they will be slightly on higher side. Indian rolled steel sections consist of sloping flanges, fillets at junctions and rounded edges. The author and K.V. Promod considered all these complexities in the shapes and determined plastic modulus of rolled steel sections and standard builtup sections and have brought out Steel Tables published by IK International Publishing House.

It may be noted that due to formation of one hinge the beam need not fail in all cases. In case of determinate structures formation of first hinge itself causes collapse of structure, since in this case it goes on rotating without resisting additional load.

If we consider a propped cantilever highly stressed section is at fixed end (Fig. 7.4) and hence plastic hinge is first formed here. After the formation of first hinge beam behaves like simply supported beam and it resists further increase in load, till one more hinge is formed. After formation of this

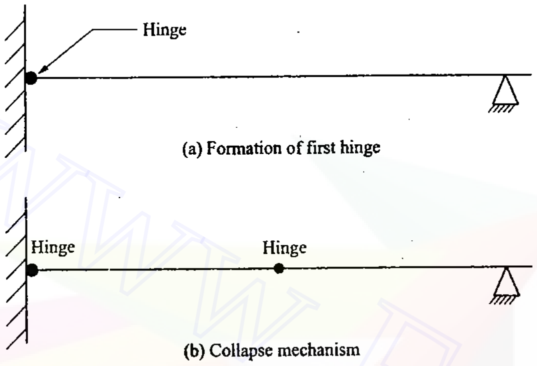

Figure 7.4

hinge the collapse mechanism is formed i.e. rotation of beam takes place without resisting any more load.

In case of fixed beam subject to symmetric loading, end hinges may form simultaneously. After that beam start acting as a simply supported beam for further load and fails only after one more hinge is formed in the middle portion. Figure 7.5 shows this case.

A frame fails only after collapse mechanism is formed. This type of analysis, known as plastic analysis is usually taught to the students in a separate course.

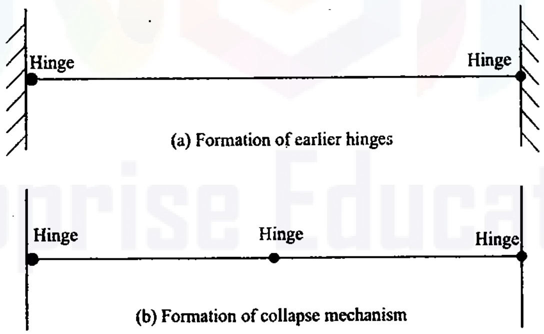

Figure 7.5

Example 7.1

Determine the plastic moment capacity and plastic section modulus of

(a) the rectangular section of size $b \times t$ about z-z axis as shown in Fig. 7.6(a).
(b) the I-section about z-z axis as shown in Fig. 7.6(b).
(c) the I-section about y-y axis as shown in Fig. 7.6(c).

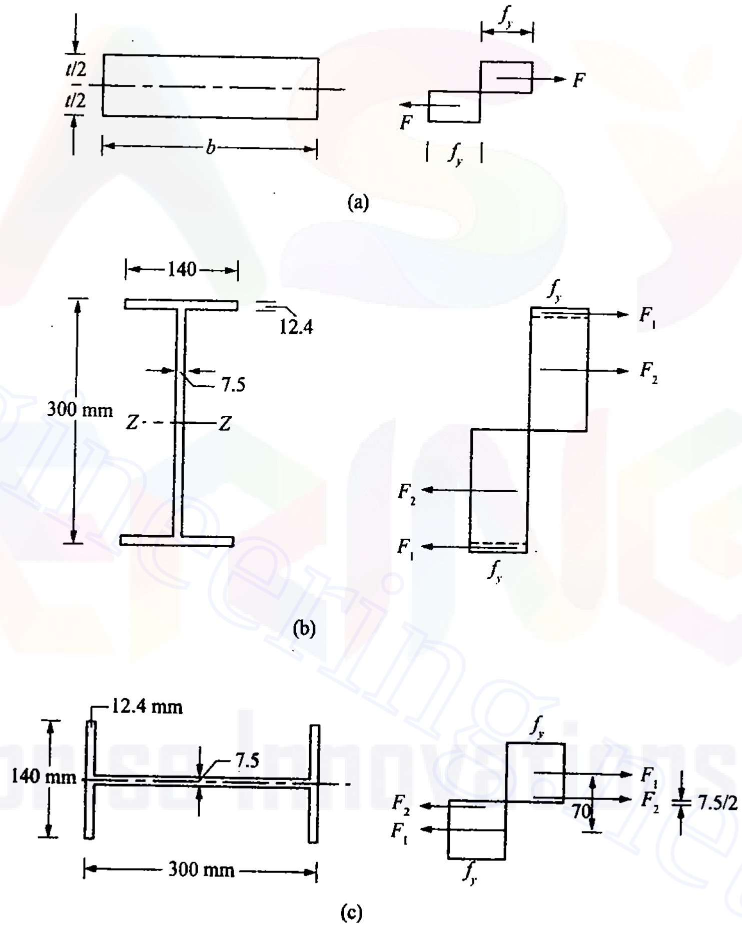

Figure 7.6

(a) *Rectangular section:*

Due to symmetry plastic neutral axis (axis of equal areas) is at mid depth.

$$\therefore \quad A_t = A_c = \frac{b}{2} \times t$$

$$F = \frac{b}{2} t\, f_y$$

The distance between tensile and compressive forces $= \frac{t}{2}$.

$$\therefore \quad M_p = F \times \frac{t}{2}$$

$$= \frac{b}{2} t\, f_y \frac{t}{2} = \frac{1}{4} b t^2 f_y$$

$$\therefore \quad Z_p = \frac{M_p}{f_y} = \frac{1}{4} b t^2$$

(b) *I-section – About z-z axis:*

Plastic N-A is at mid depth. When plastic hinge is formed

forces in flanges,

$$F_1 = 140 \times 12.4 \times f_y$$

forces in the webs

$$F_2 = \frac{1}{2}(150 - 124) \times 7.5 f_y$$

Distance between F_1 forces = 300 – 12.4 = 287.6 mm

Distance between F_2 forces = 150 – 12.4 = 137.6

$$\therefore \quad M_p = F_1 \times 287.6 + F_2 \times 137.6$$

$$= 140 \times 12.4\, f_y \times 287.6 + \frac{1}{2} \times 137.6 \times 7.5\, f_y \times 137.6$$

$$= 499274\, f_y + 71001.6\, f_y$$

$$= 570275.6\, f_y$$

$$\therefore \quad Z_p = 570.276 \times 10^3 \text{ mm}^3.$$

[*Note:* Contribution of flanges $= \frac{49274}{570276} \times 100 = 87.5\%$]

(c) *I-section about y-y axis:*

Plastic N-A is in mid depth. Let F_1 be force in flange of size 140 × 12.4 mm and F_2 be force in web of size (300 – 2 × 12.4) × 7.5 mm. Then

$$F_1 = 140 \times 12.4 f_y$$

$$F_2 = 275.2 \times 7.5 f_y$$

Distance between F_1 force $= \frac{140}{2} = 70$ mm

Distance between F_2 force $= \frac{7.5}{2} = 3.75$ mm

$$\therefore \quad M_p = 140 \times 12.4 \times f_y \times 70 + 275.2 \times 7.5 f_y \times 3.75$$

$$= 121520 f_y + 7740 f_y = 129260 f_y$$

$$\therefore \quad Z_p = \frac{M_p}{f_y} = 129260 \text{ mm}^3$$

[*Note:* Contribution of flange is $\frac{121520}{129260} \times 100 = 94\%$]

Example 7.2

Determine the plastic moment capacity and plastic modulus of section of the unsymmetric section shown in Fig. 7.7.

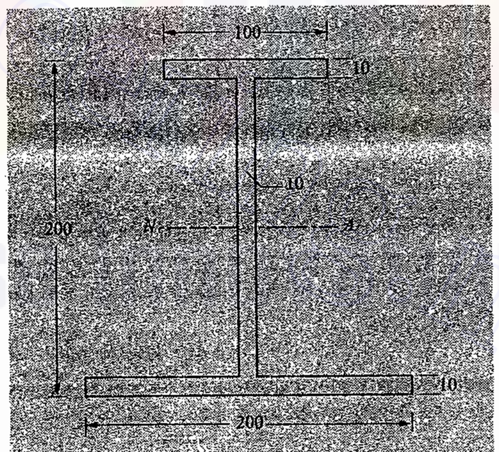

Figure 7.7

Solution:

Total area $= 100 \times 10 + 200 \times 10 + (200 - 20) \times 10 = 4800 \text{ mm}^2$

$$A_c = A_t = \frac{4800}{2} = 2400 \text{ mm}^2$$

Plastic N-A is at a depth 'h' from top fibre where h is given by

$100 \times 10 + (h - 10) \times 10 = 2400$

$\therefore \quad h = 150 \text{ mm}$

When plastic hinge is formed, one half is subjected to compressive stresses f_y and another half to tensile stresses f_y. Taking moment of all such forces about plastic NA, we get

$$M_p = \left(100\times10\times(150-5)+10\times(150-10)\frac{(150-10)}{2} + 10\times(50-10)\frac{(50-10)}{2}+200\times10\,(50-5)\right)f_y$$

$= 341000 \, f_y \text{ mm}^2$ **Answer**

$\therefore \; Z_p = \dfrac{M_p}{f_y} = 341000 \text{ mm}^3$ **Answer**

7.2 CLASSIFICATION OF CROSS-SECTIONS

When the plastic analysis is used, the members should be capable of forming plastic hinges with sufficient rotation capacity without local buckling. Hence it is necessary to see that plate elements of a cross section do not buckle locally due to compressive stresses before plastic hinges are formed. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross section subjected to compression due to axial force, moment or shear. On this basis IS: 800-2007, classifies various cross sections as follows (clause 3.7):

1. *Class 1 (Plastic) Cross-Sections:* These sections can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism. The sections having width to thickness of ratio of plate elements shall be less than that specified under class 1 as shown in Table 7.1 belong to this class.
2. *Class 2 (Compact) Cross-Sections:* Such sections can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling. The sections having width to thickness ratio of plate elements between those specified for class 2 and class 1 shown in Table 7.1 belong to this class of sections.
3. *Class 3 Cross-Sections (Semi Compact):* These are the sections in which the extreme fibre in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The sections having width to thickness ratio in the range between those shown for class 2 and class 3 in Table 7.1 belong to this class.
4. *Class 4 Cross-Sections (Slender):* The cross sections the elements of which buckle locally even before reaching yield stress belong to this category. They are having width to thickness ratio more than those specified for class 3 (ref. Table 7.1).

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IS 800-2007 considers the design of members belonging to class 4 (slender sections) as beyond its scope and hence in this book also that is treated beyond the scope. For the design of such sections reference may be made to IS 801.

Table 7.1 Limiting width to thickness ratio
Table 2 in IS 800 (*Clauses* 3.7.2 *and* 3.7.4)

Compression Element (1)		Ratio (2)	Class of Section: Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (5)
Outstanding element of	Rolled section	b/t_f	9.4ε	10.5ε	15.7ε
	Welded section	b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to	b/t_f	29.3ε	33.5ε	42ε
	Axial compression	b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth	d/t_w	84ε	105ε	126ε
	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\geq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\geq 42\varepsilon$
	If r_1 is positive:	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\geq 42\varepsilon$	
	Axial compression		Not applicable		42ε
Web of a channel		d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)		b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)		d/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member		d/t	9.4ε	10.5ε	15.7ε

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[Table 7.1 Continued]

Outstanding leg of an angle with its back in continuous contact with another component	d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I- or H-section	d/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: (a) moment	D/t	$42\varepsilon^2$	$52\varepsilon^3$	$146\varepsilon^3$
(b) axial compression	D/t	Not applicable		$88\varepsilon^3$

Notes:
1. Elements which exceed semi-compact limits are to be taken as of slender cross-section.
2. $\varepsilon = (250/f_y)^{1/2}$.
3. Webs shall be checked for shear buckling in accordance with 8.4.2. when $d/t > 67\varepsilon$, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (*see* Figs. 2, 3.7.3 and 3.7.4).
4. Different elements of a *cross-section* can be in different classes. In such cases the section is classified based on the least favourable classification.
5. The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}} \quad r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overall section}}$$

7.3 DESIGN PROCEDURE

1. A trial section is selected assuming it is going to be plastic section (class 1 section)
2. Then it is checked for the class it belongs.
3. Check for bending strength
4. Check for shear strength
5. Check for the deflection.

If any check fails the section is revised.

7.4 BENDING STRENGTH OF A LATERALLY SUPPORTED BEAM

If $\frac{d}{t_w} \le 67\,\varepsilon$. IS 800-2007 considers two cases one with design shear strength less than $0.6V_d$ and other with design shear strength more than $0.6V_d$ where V_d is design shear. When $\frac{d}{t_w} > 67\,\varepsilon$, shear buckling of web is likely to take place. For such case Ref. Art. 10.5.

(a) If $V \le 0.6V_d$:

The design bending strength M_d shall be taken as:

$$M_d = \beta_b Z_p f_y \times \frac{1}{\gamma_{mo}} \le 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}} \text{ for simply supported beam}$$

$$\le 1.5 Z_e \frac{f_y}{\gamma_{mo}} \text{ for cantilever beam}$$

where

$\beta_b = 1.0$ for plastic and compact sections

$= \frac{Z_e}{Z_p}$ for semi-compact sections.

Z_p, Z_e = plastic and elastic section modulii of the cross-section, respectively.

(b) If $V > 0.6V_d$

In such cases,

$$M_d = M_{dv}$$

where M_{dv} is design bending strength under high shear. This reduced value is recommended to account for the effect of higher shear on the bending strength of the sections. M_{dv} is to be calculated as given below (clause 9.2.2 in IS 800-2007):

(a) Plastic or Compact Section:

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \le 1.2 Z_e \times f_y \times \frac{1}{\gamma_{mo}}$$

where

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2$$

M_d = Plastic design moment of the whole section.

V = Factored applied shear force.

V_d = Design shear strength.

M_{fd} = Plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor γ_{mo}. (For finding shear area ref. Art. 7.5)

(b) Semi-Compact Section

$$M_{dv} = \frac{Z_e f_y}{\gamma_{mo}}$$

7.5 SHEAR STRENGTH OF A LATERALLY SUPPORTED BEAM

The design shear strength of a section is given by (clause 8.4 of IS 800-2007):

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{mo}}$$

where A_v = shear area

and f_{yw} = yield strength of the web.

The shear area may be calculated as given below:

(a) I and channel sections:

(i) Major Axis bending:

Hot-Rolled: $A_v = ht_w$

Welded: $A_v = dt_w$

(ii) Minor Axis bending:

Hot rolled or welded: $A_v = 2bt_f$

(b) Rectangular hollow sections of uniform thickness:

(i) Loaded parallel to depth (h): $A_v = \dfrac{Ah}{b+h}$

(ii) Loaded parallel to width (b): $A_v = \dfrac{Ab}{b+h}$

(iii) Circular hollow tubes of uniform thickness: $A_v = \dfrac{2A}{\pi}$

(iv) Plates and solid bars: $A_v = A$

where,

A = cross section area

b = overall breadth of tubular section, breadth of I-section flanges

d = clear depth of web between flanges

h = overall depth of the section

t_f = thickness of the flange and

t_w = thickness of the web.

7.6 DEFLECTION LIMITS

Deflection limits should be checked before accepting a design. In special situations the other serviceability limits like vibration limit, durability considerations and fire resistance also should be checked.

The deflection should be calculated by elastic theory for working load condition. The maximum deflection in the beam should not exceed the limits specified in Table 7.2 (Table 6 in IS 800-2007).

Table 7.2 Deflection limits
[Table 6 in IS 800]

Type of Building (1)	Deflection (2)	Design Load (3)	Member (4)	Supporting (5)	Maximum Deflection (6)
Industrial Buildings	Vertical	Live load/Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50t)	Gantry	Crane	Span/1000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane + wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Buildings	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter storey drift	–	Storey height/300

Example 7.3

A roof of a hall measuring 8 m × 12 m consists of 100 mm thick R. C. slab supported on steel I-beams spaced 3 m apart as shown in Fig. 7.8. The finishing load may be taken as 1.5 kN/m^2 and live load as 1.5 kN/m^2. Design the steel beam.

Solution:

Each beam has a clear span of 8 m and takes care of 3 m width of slab. Hence the load per metre length of the beam is as follows:

Weight of R.C. slab = 0.1 × 1 × 3 × 25 = 7.5 kN/m

Finishing load = 1.5 × 3 = 4.5 kN/m

Self weight (assumed) = 0.8 kN/m

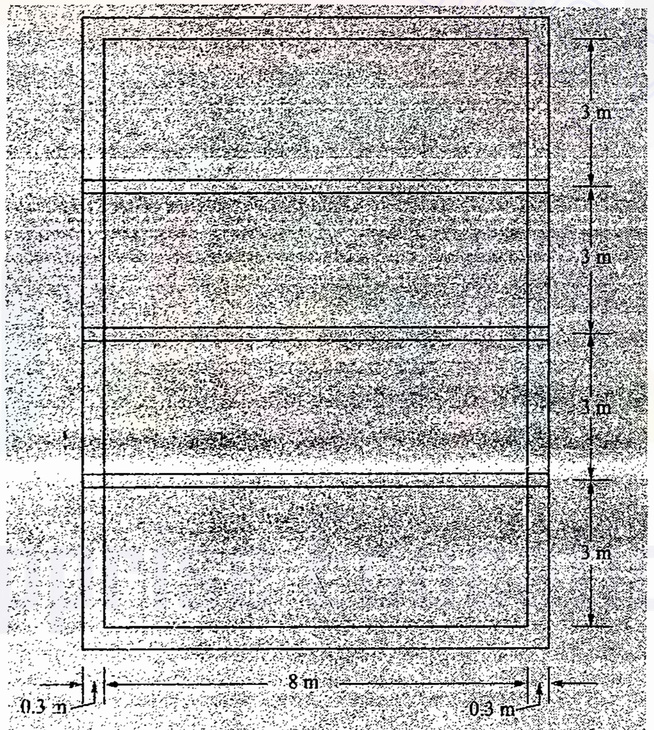

Figure 7.8

∴ Total dead load = 12.8 kN/m.

Live load = 1 × 3 × 1.5 = 4.5 kN/m.

∴ Factored dead load = 1.5 × 12.8 = 19.2 kN/m

Factored live load = 1.5 × 4.5 = 6.75 kN/m

∴ Total factored load = 25.95 kN/m.

Effective span of the simply supported beam = centre to centre distance of supports

Assuming width of support = 0.3 m,

Effective span = 8 + 0.3 = 8.3 m.

∴ Design moment, $M = \frac{wL^2}{8}$

$$= \frac{25.95 \times 8.3^2}{8} = 223.46 \text{ kN-m}$$

Design shear force $V = \frac{25.95 \times 8.3}{2} = 107.69$ kN

∴ Section modulus required $= \frac{M}{f_y} \times \gamma_{mo}$

$$Z_p = \frac{223.46 \times 10^6 \times 1.1}{250} = 983224 \text{ mm}^3$$

Try ISMB 400 which has $Z_p = 1176.163 \times 10^3$ mm^3.

The properties of the section are as follows:

Depth of section h = 400 mm

Width of flange b = 140 mm

Sectional area A = 7845.58 mm^2

Thickness of flange t_f = 16.0 mm

Thickness of web t_w = 8.9 mm

Depth of web $d = h - 2\,(h_2)$

$= 400 - 2(32.8) = 333.4$ mm

Moment of inertia about z-z axis

$I_{zz} = 20458.4 \times 10^4$ mm^4

Elastic section modulus $Z_e = 1022.7 \times 10^3$ mm^4

Outstanding leg of comp. flange, $b = \frac{140}{2} = 70$

Section Classification:

$$\epsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1.0$$

$$\frac{b}{t_f} = \frac{70}{16} = 4.38 < 9.4\,\epsilon$$

$$\frac{d}{t_w} = \frac{333.4}{8.9} = 37.57 < 84\,\epsilon$$

Hence the section is classified as plastic section:

Weight of the section = 0.604 kN/m.

Assumed weight = 0.8 kN/m.

Difference is not much. Hence the design is continued with moments and shears calculated as earlier.

Check for shear strength:

Design shear $V = 107.69$ kN

Design shear strength of the section

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times \text{shear area}$$

$$= \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 400 \times 8.9$$

$$= 467128 \text{ N} = 467.128 \text{ kN} > 107.61 \text{ kN}$$

Hence the section is adequate.

$$0.6\,V_d = 0.6 \times 467.128 = 280.277 > 107.61 \text{ kN}$$

Hence it is not high shear case.

Check for moment capacity:

$\frac{d}{t_w} = 38.2$ which is less than 67 ϵ, since $\epsilon = 1$.

Hence, $M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} \le 1.2 \frac{Ze\, f_y}{\gamma_{mo}}$

$\beta_b = 1.0$ since it is plastic section.

$$\therefore\; M_d = 1.0 \times 1176.163 \times 10^3 \times \frac{250}{1.1} \le 1.2 \times 1022.7 \times 10^3 \times \frac{250}{1.1}$$

$$= 267.310 \times 10^6 \le 278.918 \times 10^6$$

Hence adequate. $\therefore\; M_d = 267.310 \times 10^6$ N-mm $= 267.310$ kN-m.

Check for deflection:

Total working load = 12.8 + 4.5 = 17.3 kN/m.

= 17.3 N/mm

Maximum deflection

$$\delta = \frac{5}{384} \frac{wL^4}{EI}$$

$$\therefore\; \delta = \frac{5}{384} \times \frac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4}$$

$$= 26.127 \text{ mm.}$$

Permissible deflection for a beam in building (Ref. Table 7.2) = $\frac{l_e}{300} = \frac{8300}{300} = 27.67$ mm

Hence deflection is within the permissible limit.

$\therefore$ Provide ISMB 400.

Example 7.4

Design a simply supported beam of effective span 1.5 m carrying a factored concentrated load of 360 kN at mid span.

Solution:

Maximum moment occurs at mid span and is given by

$$M = \frac{WL}{4} = \frac{360 \times 1.5}{4} = 135 \text{ kN-m} = 135 \times 10^6 \text{ N-mm}$$

$\therefore$ Z_p required is obtained from the relation $f_y \frac{Z_p}{\gamma_{mo}} = M$

or

$$Z_p = \frac{135 \times 10^6}{250} \times 1.1 = 594.0 \times 10^3 \text{ mm}^3$$

Select trial section as ISMB 300 which has $Z_p = 651.731 \times 10^3$ mm^3.

The sectional properties of ISMB 300 are

Overall depth $h = 300$ mm.

Width of flange $b = 140$ mm

Thickness of flange $t_f = 12.4$ mm

$\therefore$ h_f = Centre to centre distance of flanges

$$= 300 - \frac{12.4}{2} = 293.8 \text{ mm}$$

Depth of web $d = h - 2(t_f + r_1)$

$$= 300 - 2(12.4 + 14) = 247.2 \text{ mm}$$

Thickness of web $t_w = 7.5$ mm

$$I_{zz} = 8603 \times 10^4 \text{ mm}^4$$

$$Z_e = 573.6 \times 10^3 \text{ mm}^4$$

$$Z_p = 651.73 \times 10^3 \text{ mm}^4$$

Self weight of beam = 0.4336 kN/m.

$\therefore$ Factored weight = 1.5 × 0.4336 kN/m

$\therefore$ Additional factored moment due to self at $= 1.5 \times 0.4336 \times \frac{1.5^2}{8} = 0.183$ kN-m

$\therefore$ Total factored moment

$$M = 135 + 0.183 = 135.183 \text{ kN-m.}$$

Factored shear force due to self weight $= 1.5 \times 0.4336 \times \frac{1.5}{2} = 0.488$ kN

$\therefore$ Total factored shear force on section $= \frac{360}{2} + 0.488 = 180.488$ kN

Section classification:

$$\in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1, \text{ overhang } b = \frac{140}{2}$$

$$\frac{b}{t_f} = \frac{140/2}{12.4} = 5.64 < 9.4\in$$

$$\frac{d}{t_w} = \frac{247.2}{7.5} = 32.96 < 84\in$$

It is classified as a plastic (class 1) section:

Shear capacity of the section:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5$$

$$= 295.235 \times 10^3 \text{ N} = 295.235 \text{ kN}$$

Section is adequate to resist shear

$\therefore$ $0.6\,V_d = 0.6 \times 295.235 = 177.145$ kN

$\therefore$ $V > 0.6\,V_d$

Moment capacity of the section:

Since V > 0.6 V_d and section belongs to plastic category,

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \le 1.2\, z_e f_y \times \frac{1}{\gamma_{mo}}$$

$$M_d = Z_p f_y \frac{1}{\gamma_{mo}} \le 1.2\, Z_e f_y \times \frac{1}{\gamma_{mo}}$$

Now $Z_p f_y \times \frac{1}{\gamma_{mo}} = 651.7 \times 10^3 \times 250 \times \frac{1}{1.1} = 148.114 \times 10^6$ N-mm

$$1.2\, Z_e f_y \times \frac{1}{\gamma_{mo}} = 1.2 \times 573.57 \times 10^3 \times 250 \times \frac{1}{1.1} = 156.428 \times 10^6 \text{ N-mm}$$

$\therefore$ $M_d = 148.120 \times 10^6$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2 \times 180.448}{295.235} - 1\right)^2 = 0.05$$

Since it is double symmetric section, f_{crb} may be obtained from Table 14 of the code.

$$\frac{KL}{r} = \frac{1500}{28.4} = 52.8 \text{ and } \frac{h_f}{t_f} = \frac{293.8}{12.4} = 23.69$$

Referring to Table 14, by double interpolation we get $f_{crb} = 888.2$

From Table 13 (a),

$$f_{cd} = 204.5 - \frac{88.2}{100}(206.8 - 204.5)$$

$$= 204.77$$

$\therefore$ $M_{fd} = f_{cd} \times A = 204.77 \times 5626 = 1.15204 \times 10^6$ N-mm

$\therefore$ $M_{dv} = 148.114 \times 10^6 - 0.05\,(148.114 \times 10^6 - 1.15204 - 10^6)$

$$= 140.77 \times 10^6 \text{ N-mm} = 140.77 \text{ kN-m} > 135.190 \text{ kN-m.}$$

Hence O.K.

Maximum deflection corresponding to working load

$$\delta = \frac{WL^3}{48EI} = \frac{360\times10^3\times1500^3}{48\times2\times10^5\times8603\times10^4}$$

$$= 1.68 \text{ mm} < \frac{1500}{300}$$

Hence section is adequate.

Use ISMB 300 as beam.

7.7 WEB BUCKLING STRENGTH

Certain portion of beam at supports acts as column to transfer the load from beam to the support. Hence under this compressive force the web may buckle [Ref. Fig. 7.9]. This may happen under a concentrated load on the beam also. The load dispersion angle may be taken as 45°. Hence there is need to check for web buckling. However, the rolled section are provided with suitable thickness for web so that web buckling is avoided. In case of built up sections it is necessary to check for buckling of web and provide web stiffeners (which is explained in the chapter on plate girder).

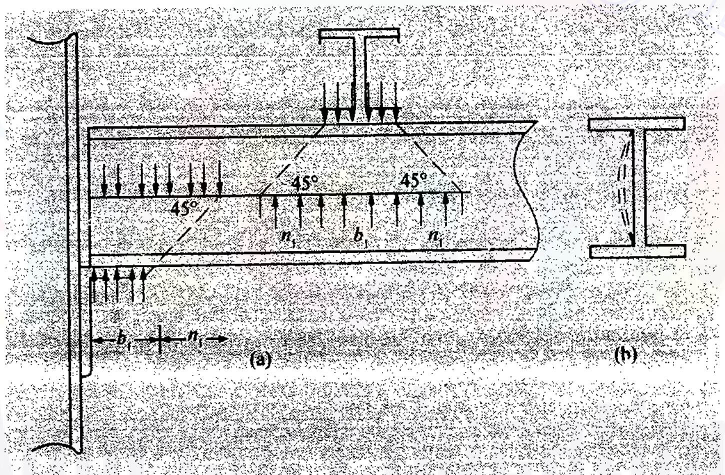

Figure 7.9

Hence as per IS 800-2007, effective web buckling strength is to be found based on the cross-section of web

$$= (b_1 + n_1)\, t_w.$$

Where b_1 = width of stiff bearing on the flange and $n_1 = \frac{1}{2}h$, where h is the depth of section.

$$\therefore \quad F_{cdw} = (b_1 + n_1)\, t_w f_c$$

where F_{cdw} – web buckling strength

and f_c is the allowable compressive stress corresponding to the assumed web column.

Effective length = 0.7d of web column

$$r_y = \sqrt{\frac{I_y}{A}} \text{ of web}$$

$$= \sqrt{\frac{\frac{1}{12}(b_1+n_1)t_w^3}{(b_1+n_1)t_w}} = \frac{t_w}{2\sqrt{3}}$$

$$\therefore \text{ Slenderness ratio} = \frac{\text{Effective length}}{r_y}\; 0.7d \cdot \frac{2\sqrt{3}}{t_w} \approx 2.5\,\frac{d}{t_w}$$

Corresponding to this slenderness ratio from Table 9 of IS 800-2007 (Table 6.4 in this book) buckling stress f_c can be found and hence

$$F_{cdw} = (b_1 + n_1)\, t_w f_c \text{ may be found.}$$

7.8 WEB CRIPPLING

Near the support web of the beam may cripple due to lack of bearing capacity as shown in the Fig. 7.10. The crippling occurs at the root of the radius. IS 800-2007 has accepted the following formula to find crippling strength of web [Ref. clause 8.7.4].

$$F_w = (b_1 + n_2)\, t_w \frac{f_{yw}}{\gamma_{mo}}$$

where,

b_1 = stiff bearing length

n_2 = length obtained by dispersion through the flange to the web junction at a slope 1:2.5 to the plane of flange

f_{yw} = yield stress of the web.

In the design F_w > Load transferred by bearing.

The care is taken in fixing the web thickness of rolled steel sections to avoid such failures. Hence if rolled steel section is selected as a beam section there is no need to check for this failure. However when built up sections are selected the web should be checked for this local failure.

Example 7.5

Check the section selected in example 7.3 for web buckling and web crippling if stiff bearing is over a length b_1 = 75 mm.

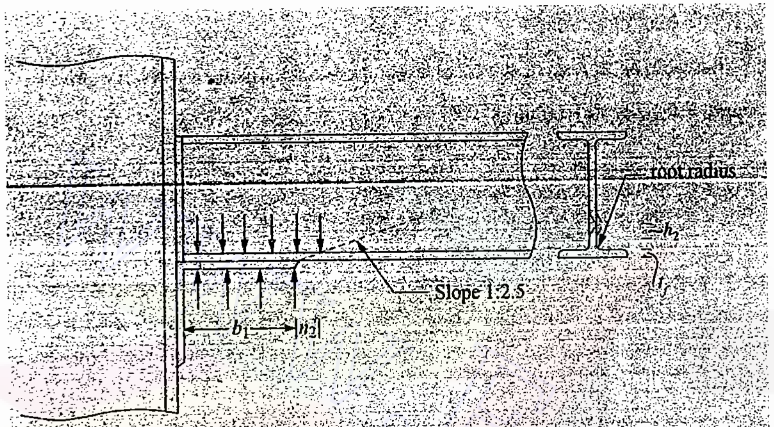

Figure 7.10

Solution:

Section selected was ISMB 400.

End reaction = End shear = 107.61 kN.

Stiff bearing at ends = 75 mm.

From steel table,

$t_w = 8.9$ mm, $t_f = 16.0$ mm,

radius at root = 14.0 mm.

Depth of section $h = 400$ mm.

$\therefore$ Depth of web $= h_1 = 334.2$

Check for web buckling:

Slenderness ratio $= \lambda \approx 2.5 \dfrac{h_1}{t_w} = \dfrac{2.5 \times 334.2}{8.9} = 93.88$

Since cross section of web is rectangle, it falls under buckling class C.
Hence from Table 9.c of IS 800-2007 (Table 6.4) we get,

$$f_c = 121 - \frac{3.88}{10}(121 - 107) = 115.568 \text{ N/mm}^2$$

$$n_1 = \frac{400}{2} = 200 \text{ mm}$$

$\therefore$ Web buckling resistance of the section,

$$F_{cdw} = (b_1 + n_1)\, t_w f_c$$

$$= (75 + 200) \times 8.9 \times 115.568 = 282.852 \times 10^3 \text{ N} = 282.852 \text{ kN} > 107.61 \text{ kN}$$

Hence the section is safe against web buckling.

Check for web crippling:

Flange thickness = 16.0 radius at root = 14.0

$\therefore$ $n_2 = 2.5\,(h_2) = 2.5 \times 32.8 = 82$ mm.

$\therefore$ Strength of web against web crippling

$$F_w = (b_1 + n_2)\, t_w\, f_{yw} \times \frac{1}{\gamma_{mo}}$$

$$= (75 + 82)\, 8.9 \times 250 \times \frac{1}{1.1} = 317.568 \times 10^3 \text{ N}$$

$= 317.568$ kN > load transferred by bearing in this case (107.61 kN).

Hence safe.

Example 7.6

Determine the uniformly distributed load carrying capacity of the welded plate girder shown in Fig. 7.11, when it is used as a cantilever beam of 4 m effective span and check it for shear, deflection, web buckling and web crippling. Assuming stiff bearing length as 100 mm.

Solution:

Section Moduli

$$I_{zz} = \frac{1}{12}\left[200 \times 832^3 - 184 \times 800^3\right] = 1748.173 \times 10^6 \text{ mm}^4$$

$$\therefore\ Z_e = \frac{I_{zz}}{y_{max}} = \frac{1748.173 \times 10^6}{\left(\dfrac{832}{2}\right)} = 4202.338 \times 10^3 \text{ mm}^3$$

Plastic N-A is at mid depth. Hence stress is f_y (comp) in top half and f_y (tensile) in bottom half

$$M_p = (200 \times 16 \times 816 + 400 \times 16 \times 400)\, f_y$$

$$\therefore\ Z_e = \frac{M_p}{f_y} = 5171.200 \times 10^3 \text{ mm}^3$$

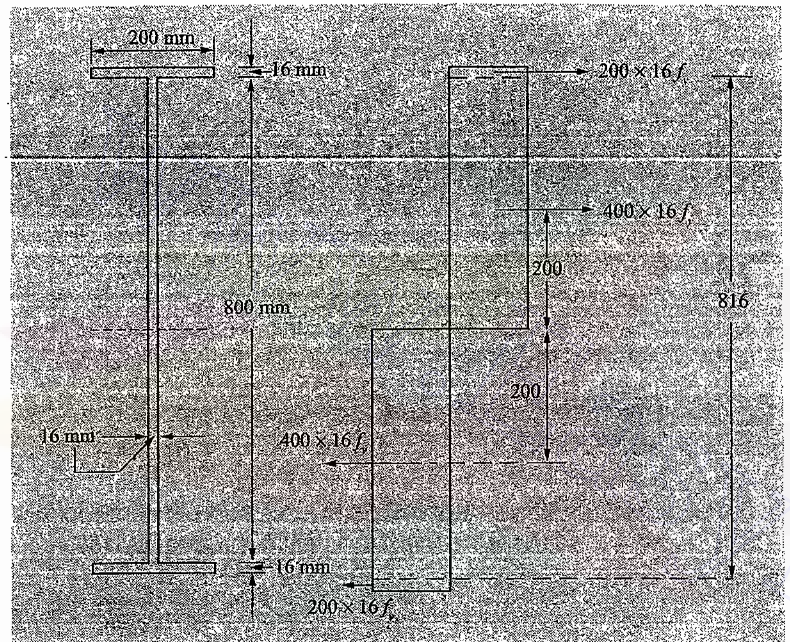

Figure 7.11

Section classification:

$$\in = 1$$

$$\frac{b}{t_f} = \frac{200}{16} = 12.5, \qquad \text{between } 9.4t \text{ and } 13.6t$$

$$\frac{d}{t_w} = \frac{800}{16} = 50 \qquad < 84 \in$$

It belongs to semicompact class of section.

Trial section:

Assuming $V < 0.6\ V_d$,

$$\therefore \quad M_d = \beta_b Z_p f_y \frac{1}{\gamma_{mo}} = \frac{Z_e}{Z_p} Z_p f_y \frac{1}{\gamma_{mo}}$$

$$= Z_e f_y \frac{1}{\gamma_{mo}}$$

$$= \frac{4202.338 \times 250}{1.1} = 955.0768 \times 10^6 \text{ N-mm} = 955.0768 \text{ kN-m}$$

Let factored *udl* be w kN per metre length. Then,

$$M = \frac{wL^2}{2} = w \times \frac{4^2}{2} = 8w$$

Equating it to M_d, we get

$$8w = 955.0768$$

$$w = 119.385 \text{ kN/m.}$$

$$\therefore \quad \text{Shear} \quad V = wL = 119.385 \times 4 = 477.538 \text{ kN.}$$

Check for shear:

Overall depth $\quad h = 800 + 16 + 16 = 832$ mm

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 832 \times 16$$

$$= 1746.747 \times 10^3 = 1746.747 \text{ kN} > V \qquad \text{safe.}$$

$$0.6\ V_d = 286.523 \text{ kN} > V$$

Hence calculated M_d is correct.

Check for deflection:

For a cantilever beam

$$\delta = \frac{wL^4}{8EI_{zz}}$$

$$w = \text{working load} = \frac{119.38 \times 1000}{1.5 \times 1000} \text{ N/mm}$$

$$= 79.587 \text{ N/mm.}$$

$$\therefore \quad \delta = \frac{79.587 \times (4000)^4}{8 \times 2 \times 10^5 \times 1748.173 \times 10^6}$$

$$= 7.28 \text{ mm} < \frac{4000}{300}$$

Hence safe.

Check for web buckling:

Since it is builtup section, h = depth of web plate = 800 mm.

Slenderness ratio $\lambda = 2.5 \dfrac{h}{t_w} = 2.5 \times \dfrac{800}{16} = 125$

Since the cross section of web is rectangle, it belongs to buckling class *C*.

From Table 9(c), $f_c = 83.7 - \frac{50}{10}(83.7 - 74.3)$

$$= 76.65 \text{ N/mm}^2$$

$$\therefore \quad F_{cdw} = (b_1 + n_1)\, t_w f_c = (100 + 416) \times 16 \times 76.65$$

$$= 632.822 \times 10^3 \text{ N}$$

$$= 632.822 \text{ kN} > V$$

$\therefore$ Web is safe.

Check for web crippling:

In this case $n_2 = 2.5\, t_f$

$$F_w = (b_1 + 2.5\, t_f)\, f_y \frac{1}{\gamma_{mo}} t_w$$

$$= (100 + 2.5 \times 16) \times 250 \frac{1}{1.1} \times 16$$

$$= 509.09 \times 10^3 \text{ N}$$

$$= 509.09 \text{ kN} > V$$

Hence safe.

$\therefore$ We conclude that the *load carrying capacity (factored) is 119.38 kN/m.*

7.9 DESIGN OF BUILT UP SECTION

When moment to be resisted is heavy, available rolled sections may not be sufficient. In such cases built up beams are used. In bolted beams the area of tension flange is reduced by bolt holes and hence the actual neutral axis moves up and I_{zz} value changes. However in the design practice, the neutral axis is still regarded as at the symmetric axis. The design procedure is illustrated with example.

Example 7.7

Design a simply supported beam of 10 m effective span carrying a total factored load of 60 kN/m. The depth of beam should not exceed 500 mm. The compression flange of the beam is laterally supported by floor construction. Assume stiff end bearing is 75 mm.

Solution:

$L = 10 \text{ m} = 10000 \text{ mm}, w = 60 \text{ kN/m}$

Trial Section:

$$\text{Maximum } BM, M = \frac{wL^2}{8} = \frac{60 \times 10^2}{8} = 750 \text{ kN-m}$$

$$= 750 \times 10^6 \text{ N-mm.}$$

$$\therefore \quad Z_p \text{ required} = \frac{M\gamma_{mo}}{f_y} = \frac{750 \times 10^6 \times 1.1}{250}$$

$$= 3300 \times 10^3 \text{ mm}^3.$$

Since depth restricted is 500 mm, select ISMB 450 and suitable plates over flanges.

Z_p of ISMB 450 = 1553.4×10^3

Z_p to be provided by cover plates

$$= (3300 - 1553.4) \times 10^3$$

$$= 1746.6 \times 10^3 \text{ mm}^3$$

If A_p is the area of cover plate on each side tensile force and compressive forces developed at the time of hinge formation = $A_p f_y$

If the distance between the two plates is 'd', plastic moment resisted = $A_p f_y d$.

Hence the additional Z_p provided by the cover plates may be taken as

$$Z_p \text{ of plates} = \frac{A_p f_y d}{f_y} \times \frac{1}{\gamma_{mo}} = \frac{A_p d}{1.1}$$

$$\therefore \quad \frac{A_p d}{1.1} = 1746.6 \times 10^3$$

Taking $d = 450 + t \approx 450$ mm.

$$\text{we get } A_p = \frac{1746.6 \times 10^3 \times 1.1}{450} = 4269.5 \text{ mm}^2$$

Provide 220 × 20 mm plates on either side.

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 450 \times 9.4$$

$$= 555.044 \times 10^3 \text{ N} = 559.044 \text{ kN}$$

$$V = 60 \times \frac{10}{2} = 300 \text{ kN}$$

Hence section is safe in shear.

Check for moment capacity:

Section classification: Outstanding element of compression flange:

$$\frac{b}{t_f} = \frac{150/2}{17.4} = 4.3 < 9.4\epsilon \text{ and}$$

$$\frac{d}{t_w} = \frac{450-2(17.4+15)}{9.4} = 40.9 < 84\epsilon$$

Hence plastic section.

Now 0.6 v_d = 0.6 × 559.044 = 0.333.026 > V

$$\therefore \quad M_d = 1.0 \times \frac{Z_p f_y}{\gamma_{mo}} < 1.2\, Z_e f_y \frac{1}{\gamma_{mo}}$$

$$\frac{Z_p f_y}{\gamma_{mo}} = \left(Z_p \text{ of I section} + Zp \text{ of plates}\right)\frac{f_y}{\gamma_{mo}}$$

$$= \left[1553.4\times10^3 + 220\times20\times(450+20)\right]\times\frac{250}{1.1}$$

$$= 823.0455 \times 10^6 \text{ N.mm}$$

$$I_{zz} = I_{zz} \text{ of ISMB 450} + I_{zz} \text{ due to plates}$$

$$= 30390.8 \times 10^4 + 2 \times A_p \left(\frac{d}{2}\right)^2$$

$$= 30390.8 \times 10^4 + 2 \times 20 \times 220\,(225 + 10)^2$$

$$= 789.888 \times 10^6 \text{ mm}^4$$

$$Z_e = \frac{789.88\times10^6}{255+20} = 3224033 \text{ mm}^4$$

$$\therefore 1.2\, Z_e F_y \frac{1}{\gamma_{mo}} = 1.2 \times 3224033 \times 250 \times \frac{1}{1.1} = 879.28 \times 10^6 \text{ N-mm}$$

$$\therefore M_d = 823.045 \times 10^6 > M$$ **Hence O.K.**

Check for deflection:

Working load $= \frac{60}{1.5}$ **kN /m = 40 kN/m = 40 N/mm**

$$\delta = \frac{5wL^4}{384\,EI_{zz}}$$

$$\therefore \delta = \frac{5\times40\times(10000)^4}{384\times2\times10^5\times789.884\times10^6} = 32.97 \text{ mm}$$

If elastic cladding is assured, permissible deflection is,

(Table 6 of IS 800), $\frac{L}{240} = \frac{10,000}{240} = 41.67$ **mm**

Hence safe.

Check for web buckling:

h_1 **= 379.2 mm.**

$$\therefore \quad \lambda = 2.5\frac{h_1}{t_w} = 2.5 \times \frac{379.2}{9.4} = 100.85$$

From Table 9.c of IS 800-2007

$$n_1 = \frac{450 + 2\times20}{2} = 490/2$$

$$f_{cd} = 107 - \frac{0.85}{10}(107 - 94.6) = 105.929 \text{ N/mm}^2$$

$$F_{cdw} = (b_1 + n_1)\, t_w\, f_{cd} = \left(75 + \frac{490}{2}\right) 9.4 \times 84.05$$

$$= 318.634 \times 10^3 \text{ N} > 300 \text{ kN.}$$

∴ **Safe.**

Check for web crippling:

$$F_w = \left(b_1 + 2.5t_f\right) f_y \frac{1}{\gamma_{mo}} t_w$$

$$= [175 + 2.5\times(17.4+20)]\, 250 \times \frac{1}{1.1} \times 9.4$$

$$= 573.614 \times 10^3 \text{ N} > 300 \text{ kN}$$

Hence safe.

Design of connection between flange plates and flange:

Bolts/welds joining the plates and flange are to be designed for the horizontal shear at that level.

Shear stress at the level of plates and flanges

$$= \frac{F}{bI_{zz}}(a\bar{y})$$

$$= \frac{300 \times 10^3}{150 \times 789.884 \times 10^6}(220 \times 20 \times 225)$$

$$= 2.506 \text{ N/mm}^2$$

If bearing type bolts are used, strength in single shear = $\frac{f_u}{\sqrt{3}} \times 0.78 \frac{\pi}{4} d^2 \times \frac{1}{1.25}$

Using 16 mm bolts

Strength in single shear = $\frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2 \times \frac{1}{1.25}$

$= 28974$ N

Strength in bearing is more, if minimum specified edge distances are provided. There are two bolts in a pitch distance (one on either side of web). Hence shear force per pitch length = shear strength of 2 bolts

i.e., $p \times (150 \times 2.506) = 2 \times 28974$

$p = 154.16$ mm

Provide 16 mm bolts at 150 mm c/c.

[*Note*: As per elause 10.2.3.2 in IS 800-2007, the distance between the centres of any two adjacent fasteners in a line lying in the direction of stress shall not exceed 16*t* or 200 mm whichever is less in tension members and 12*t* or 200 mm whichever is less, in compression members].

7.10 DESIGN STRENGTH OF LATERALLY UNSUPPORTED BEAMS

Beams are normally used so as to bend about major (*z-z*) axis rather than to bend about minor (*y-y*) axis, since they have higher value of moment of inertia about that axis. In such cases when the compression flange is not supported, it has a tendency to bend in the lateral direction with twisting. [Ref. Fig. 7.12.]

Bending of compression flange with twisting reduces the load carrying capacity of the section.

To reduce the effect of lateral buckling beams are invariably provided with restraints at ends and some intermediate points. Restraint against torsional rotation at supports may be provided by

(a) building the beam into the walls or
(b) web or flange cleats or
(c) bearing stiffeners acting in conjunction with the bearing of the beams or
(d) lateral end frames or external supports providing lateral restraint to the compression flanges at the ends.

Intermediate lateral supports are provided by bracing system or by connecting to an independent robust post of structure. Such restraint should be capable of resisting 2.5 per cent of the maximum force in the compression flange and should be connected closer to compression flanges.

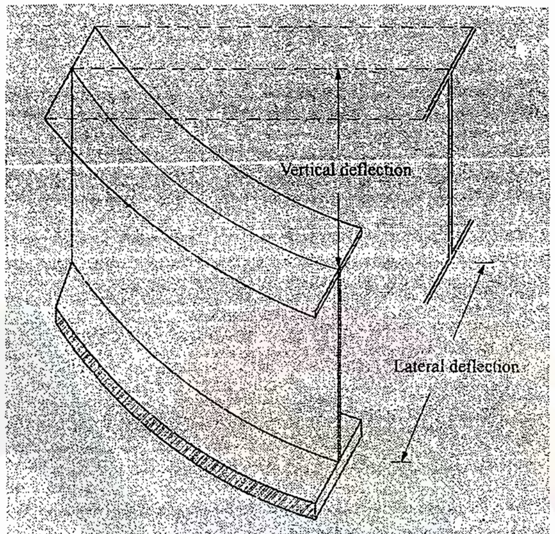

Figure 7.12 Torsional buckling of beams.

IS 800-2007, specify that the member may be treated as laterally supported in the following cases:

(a) Bending is about the minor axis
(b) Section is hallow (rectangular / circular) or solid bar and
(c) Non-dimensionalised slenderness ratio γ_{LT} is less than o.4, where

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}}$$

The elastic critical buckling stress f_{cr} is to be found considering torsional and flexural rigidities of the member as specified by clause 8.2.2.1 in IS 800-2007. For different beam sections; considering loading, support conditions and non-symmetric section shall be more accurately calculated using the method given in Annex E of IS code. For prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections. IS 800 gives values of f_{crb} based on the values of slenderness ratio and the ratio of $\frac{h}{t_f}$ in Table 14 (Table 7.3).

Taking imperfection factors $\alpha_{LT} = 0.21$ for rolled steel sections and $\alpha_{LT} = 0.49$ for welded steel section. IS 800 has prepared Tables 13.a (Table 7.4a) and 13.6b (Table 7.4b) for the benefit of users to determine

Table 7.3 Critical stress, f_{crb}
[Refer Table 14 in IS 800]

	h_f/t_f														
KL/r	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
10	22 551.2	22 255.1	22 092.6	21 994.1	21 929.8	21 885.7	21 854.0	21 805.4	21 779.0	21 763.1	21 752.7	21 740.5	21 733.8	21 727.2	21 724.2
20	6 220.5	5 947.9	5 794.5	5 700.0	5 637.8	5 594.7	5 563.8	5 515.8	5 489.7	5 473.8	5 463.5	5 451.4	5 444.8	5 438.2	5 435.1
30	3 149.3	2 905.9	2 764.6	2 676.0	2 616.7	2 575.3	2 545.3	2 498.5	2 472.8	2 457.1	2 447.0	2 434.9	2 428.3	2 421.7	2 418.6
40	2 036.1	1 821.2	1 693.0	1 610.8	1 555.1	1 515.8	1 487.0	1 441.7	1 416.5	1 401.1	1 391.0	1 379.0	1 372.5	1 365.9	1 362.8
50	1 492.9	1 303.2	1 187.3	1 111.8	1 059.9	1 022.7	995.3	951.7	927.1	912.0	902.0	890.2	883.7	877.1	874.2
60	1 178.0	1 009.5	905.0	835.6	787.4	752.4	726.4	684.6	660.9	646.1	636.4	624.7	618.2	611.7	608.7
70	973.9	823.2	728.5	664.8	620.1	587.4	562.9	522.9	500.0	485.5	476.0	464.4	458.0	451.7	448.7
80	831.3	695.4	609.2	550.7	509.1	478.4	455.3	417.2	395.1	381.2	371.8	360.5	354.1	347.7	344.7
90	725.9	602.6	523.6	469.5	430.9	402.2	380.4	344.2	322.9	309.3	300.2	289.1	282.8	276.5	273.5
100	644.7	532.0	459.3	409.3	373.2	346.4	325.8	291.4	270.9	257.7	248.8	237.9	231.8	225.5	222.5
110	580.4	476.6	409.3	362.9	329.2	303.9	284.5	251.8	232.1	219.3	210.8	200.1	194.0	187.8	184.8
120	527.9	431.9	369.5	326.0	294.5	270.7	252.3	221.2	202.4	190.1	181.6	171.2	165.2	159.1	156.2
130	484.3	395.0	336.8	296.1	266.5	244.1	226.7	197.1	179.0	167.1	158.8	148.6	142.8	136.7	133.9
140	447.6	364.2	309.5	271.5	243.4	222.3	205.8	117.5	160.2	148.7	140.7	130.8	125.0	119.0	116.2
150	416.0	337.8	286.6	250.6	224.2	204.2	188.4	161.5	144.8	133.7	126.0	116.3	110.6	104.7	101.9
160	388.7	315.2	266.8	232.8	207.8	188.8	173.9	148.2	132.0	121.3	113.9	104.3	98.8	93.0	90.1
170	364.9	295.4	249.6	217.5	193.7	175.6	161.4	136.7	121.3	111.0	103.6	94.4	89.0	83.2	80.4
180	343.9	278.0	234.6	204.1	181.5	164.2	150.6	127.1	112.2	102.2	95.2	86.0	80.7	75.0	72.3
190	325.2	262.6	221.3	192.3	170.7	154.2	141.2	118.6	104.3	94.6	87.8	79.0	73.7	68.1	65.3
200	308.3	248.8	209.6	181.7	161.2	145.4	133.0	111.3	97.5	88.1	81.4	72.8	67.8	62.2	59.5
210	293.3	236.5	198.9	172.4	152.7	137.6	125.7	104.8	91.5	82.4	75.9	67.5	62.6	57.1	54.5
220	279.5	225.3	189.3	163.9	145.1	130.6	119.1	99.0	86.2	77.4	71.2	62.9	58.1	52.7	50.1
230	267.1	215.2	180.7	156.3	138.2	124.3	113.3	93.9	81.5	72.9	66.9	58.9	54.1	48.8	46.2
240	255.8	205.8	172.8	149.4	132.0	118.6	108.0	89.3	77.2	69.0	63.1	55.3	50.6	45.4	42.8
250	245.3	197.3	165.6	143.0	126.3	113.4	103.2	85.1	73.5	65.5	59.7	52.1	47.5	42.4	39.8
260	235.7	189.5	159.0	137.3	121.1	108.7	98.8	81.3	70.1	62.3	56.7	49.3	44.8	39.7	37.2
270	226.8	182.3	152.8	131.9	116.3	104.3	94.7	77.9	67.0	59.4	53.9	46.8	42.2	37.3	34.8
280	218.6	175.7	147.2	126.9	111.9	100.2	91.1	74.7	64.1	56.8	51.5	44.4	40	35.2	32.7
290	210.9	169.4	141.9	122.3	107.8	96.6	87.7	71.8	61.5	54.3	49.2	42.2	38.1	33.2	30.8
300	203.8	163.7	137.1	118.1	104.1	93.2	84.5	69.1	59.1	52.1	47.1	40.4	36.2	31.5	29.0

Table 7.4(a) Design bending compressive stress corresponding to lateral buckling, f_{bd}; $\alpha_{LT} = 0.21$
[Refer Table 13(a) in IS 800]

	f_y																		
$f_{\alpha\beta}$	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
8 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
6 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
4 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
2 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
1 000	169.1	179.5	186	196.5	202.9	209.1	219.8	229.1	245.5	261.8	275.1	291.3	300.5	323.6	332.2	355.9	370.9	384.8	412.4
900	169.1	179.5	186	195.4	200.7	204.5	215.1	231.6	242.7	258.9	272	291.3	300.5	316.4	328.4	339.5	366.5	380.2	392.7
800	167.3	177.5	184	190.3	196.4	206.8	212.7	224	240	258.9	268.9	284.7	293.6	301.8	324.5	335.5	349.1	370.9	387.8
700	163.6	171.8	182	188.2	192	202.3	208	226.5	237.3	250.2	259.6	278.2	286.7	294.5	305.5	327.3	340.4	352.4	363.3
600	161.8	168	176	181.9	194.2	197.7	203.3	218.9	226.4	244.4	253.5	261.8	276.4	287.3	294	306.8	322.9	333.8	343.6
500	161.8	166.1	172	179.8	185.5	188.6	200.9	208.7	218.2	232.7	244.2	248.7	259.1	269.1	274.9	286.4	296.7	301.4	314.2
450	158.2	164.2	168	173.5	183.3	186.4	191.5	206.2	215.5	224	231.8	242.2	248.7	258.2	263.5	274.1	279.3	292.1	294.5
400	150.9	162.3	166	169.4	174.5	184.1	186.7	196	204.5	215.3	222.5	229.1	238.4	243.6	248.2	257.7	261.8	264.3	274.9
350	147.3	152.7	162	165.2	170.2	172.7	179.6	188.4	193.6	200.7	210.2	212.7	221.1	225.5	229.1	233.2	240	241.1	245.5
300	143.6	147	152	154.7	161.5	163.6	167.8	175.6	182.7	186.2	194.7	196.4	196.9	203.6	206.2	212.7	213.8	217.9	220.9
250	134.5	137.5	142	144.3	148.4	152.3	153.6	160.4	163.6	165.8	170	173.5	179.6	178.2	179.5	184.1	183.3	185.5	191.5
200	121.8	124.1	126	129.6	130.9	134.1	134.7	137.5	141.8	142.5	145.3	147.3	148.5	149.1	152.7	151.4	152.7	153	157.1
150	101.8	103.1	104	104.5	106.9	106.8	108.7	109.5	111.8	113.5	114.4	114.5	117.5	116.4	118.4	118.6	117.8	120.5	122.7
100	74.5	76.4	76	77.4	76.4	77.3	78	78.9	79.1	78.5	80.4	81.8	79.5	80	80.2	81.8	82.9	83.5	83.5
90	67.3	68.7	70	69	69.8	70.5	70.9	71.3	70.9	72.7	74.2	72	72.5	72.7	72.5	73.6	74.2	74.2	73.6
80	61.8	63	62	62.7	63.3	63.6	63.8	63.6	65.5	64	64.9	65.5	65.6	65.5	64.9	65.5	65.5	64.9	68.7
70	54.5	55.4	56	56.5	56.7	56.8	56.7	56	57.3	58.2	58.7	58.9	58.7	58.2	57.3	57.3	56.7	60.3	58.9
60	47.3	47.7	48	48.1	48	50	49.6	48.4	49.1	49.5	49.5	49.1	48.4	50.9	49.6	49.1	52.4	51	49.1
50	40	40.1	40	41.8	41.5	40.9	40.2	40.7	40.9	40.7	43.3	42.5	41.5	43.6	42	40.9	43.6	41.7	44.2
40	32.7	32.5	32	33.5	32.7	34.1	33.1	33.1	32.7	34.9	34	32.7	34.5	32.7	34.4	32.7	34.9	32.5	34.4
30	25.5	24.8	26	25.1	26.2	25	26	25.5	24.5	26.2	24.7	26.2	24.2	25.5	26.7	24.5	26.2	27.8	24.5
20	16.4	17.2	18	16.7	17.5	18.2	16.5	17.8	16.4	17.5	18.5	16.4	17.3	18.2	19.1	16.4	17.5	18.5	19.6
10	9.1	9.5	8	8.4	8.7	9.1	9.5	7.6	8.2	8.7	9.3	9.8	10.4	7.3	7.6	8.2	8.7	9.3	9.8

Table 7.4(b) Design bending compressive stress corresponding to lateral buckling, f_{bd}, $\alpha_{LT} = 0.49$
[Refer Table 13(b) in IS 800]

$f_{cr,b}$	f_y 200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
8 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
6 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
4 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
2 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
1 000	150.0	164.2	170	179.8	185.5	190.9	196.2	211.3	220.9	235.6	247.3	255.3	266.0	280	290.2	302.7	318.5	329.2	343.6
900	154.5	164.2	170	173.5	183.3	188.6	193.8	203.6	218.2	226.9	238.0	252.0	262.5	269.1	282.5	290.5	305.5	319.9	333.8
800	152.7	158.5	168	171.5	176.7	181.8	191.5	201.1	210	224.0	234.9	242.2	252.2	258.2	271.1	282.3	296.7	306	3191.1
700	150.9	154.6	160	169.4	172.4	177.3	182.0	196	207.3	215.3	222.5	232.4	238.4	247.3	259.6	270	279.3	292.1	304.4
600	145.5	150.8	154	161.0	168.0	172.7	177.3	188.4	193.6	203.6	213.3	222.5	228	236.4	244.4	253.6	261.8	273.5	274.9
500	140.0	145.1	150	154.7	159.3	161.4	167.8	175.6	185.5	192	200.9	206.2	214.2	218.2	225.3	229.1	240	245.7	250.4
450	134.5	141.3	144	148.5	152.7	156.8	160.7	168	177.3	186.2	191.6	196.4	203.8	210.9	213.8	220.9	231.3	236.5	235.6
400	129.1	135.5	138	142.2	148.4	150.0	153.6	162.9	169.1	174.5	182.4	183.3	193.5	196.4	202.4	208.6	209.5	217.9	220.9
350	123.6	129.8	132	135.9	139.6	143.2	148.9	152.7	158.2	162.9	170	173.5	176.2	181.8	183.3	192.3	196.4	199.4	206.2
300	118.2	122.2	126	129.6	130.9	134.1	137.1	142.5	147.3	154.2	157.6	157.1	162.4	167.3	168	175.9	178.9	180.8	181.6
250	109.1	112.6	116	117.1	120.0	122.7	125.3	129.8	130.9	136.7	139.1	140.7	145.1	149.1	148.9	151.4	152.7	157.6	157.1
200	98.2	101.2	102	104.5	104.7	109.1	108.7	112	117.3	119.3	120.5	121.1	124.4	127.3	126	130.9	130.9	129.8	132.5
150	83.6	84.0	86	87.8	89.5	88.6	89.8	91.6	95.5	96.0	95.8	98.2	100.2	101.8	103.1	102.3	104.7	106.6	103.1
100	63.6	63.0	64	64.8	65.5	55.9	66.2	68.7	68.2	68.8	71.1	68.7	69.1	72.7	72.5	73.6	74.2	74.2	73.6
90	58.2	57.3	60	58.5	61.1	61.4	61.5	61.1	62.7	64.0	64.9	65.5	65.6	65.5	64.9	65.5	65.5	64.9	68.7
80	52.7	53.5	54	54.4	54.5	54.5	54.4	56	57.3	58.2	58.7	58.9	58.7	58.2	61.1	61.4	61.1	60.3	58.9
70	47.3	47.7	48	48.1	48.0	50.0	49.6	50.9	49.1	49.5	52.5	52.4	51.8	50.9	53.5	53.2	52.4	55.6	54.0
60	41.8	42.0	42	41.8	43.6	43.2	42.5	43.3	43.6	43.6	43.3	45.8	44.9	47.3	45.8	45.0	48.0	46.4	49.1
50	36.4	36.3	36	35.5	37.1	36.4	37.8	38.2	38.2	37.8	37.1	39.3	38.0	40.0	38.2	40.9	39.3	37.1	39.3
40	29.1	30.5	30	29.3	30.5	29.5	30.7	30.5	30.0	32.0	30.9	32.7	31.1	32.7	30.5	32.7	30.5	32.5	34.4
30	23.6	22.9	22	23.0	24.0	22.7	23.6	22.9	24.5	23.3	24.7	22.9	24.2	25.5	22.9	24.5	26.2	23.2	24.5
20	16.4	15.3	16	16.7	15.3	15.9	16.5	15.3	16.4	17.5	15.5	16.4	17.3	18.2	15.3	16.4	17.5	18.5	14.7
10	9.1	7.6	8	8.4	8.7	9.1	9.5	7.6	8.2	8.7	9.3	9.8	6.9	7.3	7.6	8.2	8.7	9.3	9.8

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the reduced bending strength f_{bd} of the section. Then the design bending strength of laterally unsupported beam as governed by torsional buckling is given by

$$M_d = \beta Z_p f_{bd}$$

where,

$\beta = 1.0$ for plastic and compact section

$= \dfrac{Z_e}{Z_p}$ for semi compact section.

7.11 EFFECTIVE LENGTH FOR LATERAL TORSIONAL BUCKLING

For *simply supported beams* and girders of span length L, where no lateral restraint to the compression flanges is provided but end of the beam is restrained against torsion, the effective length L_{LT} shall be taken as in Table 15 in IS 800 (Table 7.5).

In Table 7.5 (Table 15 in IS 800), normal loading means load acting through shear centre and destabilizing loading means the load acting is not through shear centre and it creates destabilizing effect.

Table 7.5 Effective length for simply supported beams, L_{LT}
[Refer Table 15 in IS 800] (*Clause* 8.3.1)

Sl No. (1)	Conditions of Restraint at Supports: Torsional Restraint (2)	Warping Restraint (3)	Loading Condition: Normal (4)	Destabilizing (5)
(i)	Fully restrained	Both flanges fully restrained	0.70 L	0.85 L
(ii)	Fully restrained	Compression flange fully restrained	0.75 L	0.90 L
(iii)	Fully restrained	Both flanges fully restrained	0.80 L	0.95 L
(iv)	Fully restrained	Compression flange partially restrained	0.85 L	1.00 L
(v)	Fully restrained	Warping not restrained in both flanges	1.00 L	1.20 L
(vi)	Partially restrained by bottom flange support connection	Warping not restrained in both flanges	1.0 L + 2 D	1.2 L + 2 D
(vii)	Partially restrained by bottom flange bearing support	Warping not restrained in both flanges	1.2 L + 2 D	1.4 L + 2 D

Notes:

1. Torsional restraint prevents rotation about the longitudinal axis.
2. Warping restraint prevents rotation of the flange in its plane.
3. D is the overall depth of the beam.

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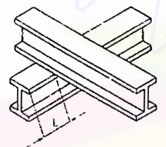
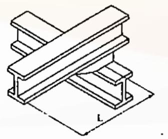
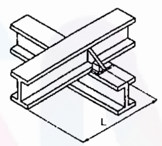
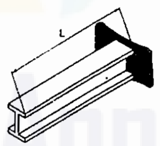
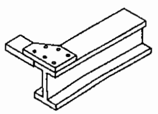
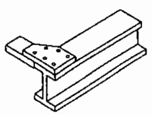
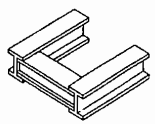
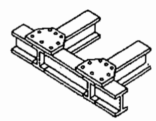
Table 7.6 [Refer Table 16 in IS 800]

Restraint Condition		Loading Condition	
At Support	At Top	Normal	Destabilizing
(1)	(2)	(3)	(4)
(a) Continuous, with lateral restraint to top flange 	(i) Free (ii) Lateral restraint to top flange (iii) Torsional restraint (iv) Lateral and torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
(b) Continuous, with partial torsional restraint 	(i) Free (ii) Lateral restraint to top flange (iii) Torsional restraint (iv) Lateral and torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
(c) Continuous, with lateral and torsional restraint 	(i) Free (ii) Lateral restraint to top flange (iii) Torsional restraint (iv) Lateral and torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
(d) Restrained laterally, torsionally and against rotation on plan 	(i) Free (ii) Lateral restraint to top flange (iii) Torsional restraint (iv) Lateral and torsional restraint	0.8L 0.7L 0.6L 0.5L	1.4L 1.4L 0.6L 0.5L
Top restraint conditions			
(i) Free 	(ii) Lateral restraint to top flange 	(iii) Torsional restraint 	(iv) Lateral and torsional restraint 

In between the lateral restraints, the effective length of the relevant segment is to be taken as 1.2 times the length of the segments.

For *cantilever beams* of projecting length L, the effective length L_{LT} to be used shall be taken as shown in Table 16 in IS 800 (Table 7.6) for different support conditions.

Example 7.8

An ISMB 500 section is used as a beam over a span of 6 m, with simply supported ends. Determine the maximum factored uniformly distributed load that the beam can carry if the ends are restrained against torsion but compression flange is laterally unsupported.

Solution:

For ISMB 500,

overall depth $h = 500$ mm

width of flange $b = 180$ mm

Thickness of flange $t_f = 17.2$ mm

Thickness of web = 10.2 mm

$r_{yy} = 35.2$ mm

Effective length for torsional buckling = 6 m.

$$\therefore \frac{KL}{r} = \frac{6 \times 1000}{35.2} = 170.45, \text{ where } KL = \text{effective length}$$

$$\frac{h}{t_f} = \frac{500}{17.2} = 29.06$$

From Table 14 of IS 800 (Table 7.3), we get

f_{crb} values as shown below:

$\frac{h}{t_f} \rightarrow$ $\frac{kL}{r}$ ↓	25	29.6	30
170	136.7	X	121.3
170.45	...	O	
180	127.1	Y	112.2

To get the value for $\frac{h}{t_f} = 29.6$ and $\frac{KL}{r} = 170.45$ it needs double linear interpolation.

First get the values at X and Y corresponding to $\frac{h}{t_f} = 29.6$

To get the value at $X\left(\frac{KL}{r} = 170, \frac{h}{t_f} = 29.6\right)$

$$f_{crb} = 136.7 - \frac{4.6}{5}(136.7 - 121.3) = 122.53 \text{ N/mm}^2$$

To get the value at $Y\left(\frac{KL}{r} = 180, \frac{h}{t_f} = 29.6\right)$

$$f_{crb} = 127.1 - \frac{4.6}{5}(127.1 - 112.2) = 113.39 \text{ N/mm}^2$$

To get the value at O.

$$f_{crb} = 122.53 - \frac{0.45}{10}(122.53 - 113.39) = 122.119 \text{ N/mm}^2$$

Referring to Table 13(a) in IS 800 [7.4(a)], for $f_y = 250$ N/mm^2,

we find $f_{bd} = 77.3$ N/mm^2 for $f_{crb} = 100$

and $f_{bd} = 106.8$ N/mm^2 for $f_{crb} = 150$.

$\therefore$ For $f_{crb} = 122.12$,

$$f_{bd} = 77.3 + \frac{22.12}{50}(106.8 - 77.3)$$

$$= 90.35 \text{ N/mm}^2$$

Section classification:

$$\in = \sqrt{\frac{250}{250}} = 1.0$$

Width of outstanding leg, $b = 180/2 = 90$ mm.

$$\frac{b}{t_f} = \frac{90}{17.2} = 5.23 < 9.4\in$$

$$d = h_1 + 2r_1 = 424.1 + 2 \times 17 = 458.1 \text{ mm}$$

$$\therefore \quad \frac{d}{t_w} = \frac{458.1}{10.2} \leq 84\in$$

Hence it belongs to class 1 (plastic) category.

$$\therefore \quad M_d = \beta_b Z_p f_{bd}$$

$$\beta_b = 1, \quad Z_p = 2074.7 \times 10^3 \text{ mm}^3, \quad f_{bd} = 90.35 \text{ N/mm}^2$$

$$\therefore \quad M_d = 1 \times 2074.7 \times 10^3 \times 90.35 = 187.449 \times 10^6 \text{ N-mm}$$

$$= 187.009 \text{ kN-m}$$

If *udl* w is in kN/m, then $\frac{wL^2}{8} = M_d$

$$w \times \frac{6^2}{8} = 187.009$$

$$w = 41.655 \text{ kN/m.} \qquad \textbf{Answer}$$

Self weight = 86.9 kg/m = 86.9 × 9.81 = 852 N/m

= 0.852 kN/m.

$\therefore$ Factored self weight = 1.5 × 0.852 = 1.278 kN/m.

$\therefore$ Super imposed *udl* that beam can carry

= 41.665 – 1.278 = 40.377 kN/m

[*Note:* Checks may be applied for shear, deflection, web buckling and web crippling to ensure that the above load can be carried safely.]

7.12 DESIGN OF LATERALLY UNSUPPORTED BEAMS

Design is possible only by trial. First f_{bd} is to be assumed to find required Z_p of the section. After selecting the trial section, calculations are made to find moment carrying capacity of the section. If the section selected is not adequate larger section is to be tried. If the section selected is having moment carrying capacity too high, resulting into uneconomical section, lower section is to be tried. For the selected section all checks are to be applied. If any check fails new section is to be tried.

7.13 DESIGN OF PURLINS

Purlin is a member which rest between roof trusses and supports roof sheeting [Fig. 7.13]. I-sections channels, angles, cold formed C or Z-sections are commonly used as purlins. The purlins are spaced on main rafter of trusses such that the sheets overlap on them. For A.C sheets the overlap of at least 150 mm is to be ensured. Hence one can see in most of the building purlins are spaced at 1.35 m to 1.40 m when A.C sheets are of length 1.65 m.

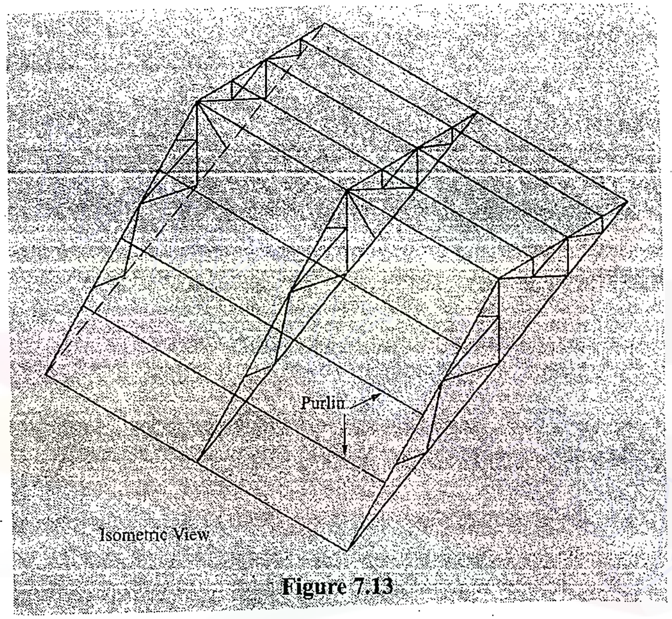


Figure 7.13

Purlins are connected to main rafter of the trusses with their web normal. Cleat angles are used for the connection. Hence the *z-z* and *y-y* axes of purlins are also inclined to vertical and horizontal. If θ is the inclination of main rafter to the horizontal then the component of dead load and live load on purlin in directions normal and perpendicular to sheeting are $W\cos\theta$ and $W\sin\theta$ respectively.

Wind load is always taken as normal to the rafter. Hence it is having bending only about z-z axis of purlins.

In case of channel and angle sections the force is not acting through the shear centre. Hence there is destabilizing effect of forces acting in *y*-direction of the purlins.

The bending of purlins takes place about both *y-y* and *z-z* axes. Hence the purlins are subjected to biaxial bending. It is a desirable point to think whether one can assume that sheets take the component of force in the direction of sheet and design the purlins for bending about *z-z* axis only.

The effective length of purlins may be taken as centre to centre distance between the supports and assume that purlins act as beams. However many times continuous purlins are also used. In case of simply supported purlins bending moment is $\frac{wL^2}{8}$ while in case of continuous purlins it may be taken as $\frac{wL^2}{10}$, where w is the load intensity.

7.14 DESIGN PROCEDURE

The following procedure is to be followed step by step in the design of purlins.

(1) Resolve the factored forces parallel to and perpendicular to sheeting.
(2) Determine moments and shear forces about *z-z* and *y-y* axes (M_z, F_z, M_y and F_y).
(3) To account for biaxial bending, the required value of section modulus about *z-z* axis may be taken as

$$Z_{pz} = \frac{M_z}{f_y}\gamma_{mo} + 2.5\frac{d}{b}\cdot\frac{M_y}{f_y}\gamma_{mo}$$

where,

γ_{mo} is partial safety factor for material = 1.1,

d is the depth of trial section and,

b is the breadth of trial section.

The second term in the expression above makes an approximate relation between Z_{pz} and Z_{py}. Since d and b values will be known only after a section is selected, first trial section is selected and Z_p required is found. If it is sufficient proceed further otherwise try another section.

(4) Check the section for shear capacity, for which the following expressions can be used.

$$V_{dz} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} A_{vz}$$

$$V_{dy} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} A_{vy}$$

where, $A_{vz} = ht_w$

and $A_{vy} = 2b_f t_f$

(5) Compute the design capacity of the section in both axes.

$$M_{dz} = \frac{Z_{pz} f_y}{\gamma_{mo}} \le 1.2 Z_{ez} \frac{f_y}{\gamma_{mo}}$$

$$M_{dy} = \frac{Z_{py} f_y}{\gamma_{mo}} \le 1.5 Z_{ey} \frac{f_y}{\gamma_{mo}}$$

(6) The design should satisfy the following interaction formula:

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \le 1.0$$

(7) Apply check for deflection.

(8) Check for wind suction: When dead load and live loads are predominant, top flange of purlin is under compression. But this flange is laterally restrained by sheeting. Hence for those loads analysis as laterally restrained compression flange was adequate. But when wind suction acts, bottom flange will be in compression which is not restrained laterally. Obviously in this case wind suction along with dead load only should be considered as critical load. Considering torsional buckling M_{dz} and M_{dy} should be found and the interaction formula should be checked.

Example 7.9

Symmetric trusses of span 20 m and height 5 m are spaced at 4.5 m centre to centre. Design channel section purlins to be placed at 1.4 m distances to resist the following loads:

Weight of sheeting including bolts = 171 N/m^2

Live load = 0.4 kN/m^2

Wind load = 1.2 kN/m^2, suction

Solution:

Height of truss = 5 m   Span of truss = 20 m

∴ Slope of main rafter of symmetric truss

$$\tan\theta = \frac{5}{10} \quad \text{or} \quad \theta = 26.565$$

Design for DL + LL:

D L from sheeting = 171 N/m^2

Self weight of purlins = 125 N/m^2.   (assumed)

∴ Total dead load = 171 + 125 = 296 N/m^2 = 0.296 kN/m^2

Live load = 0.4 kN/m^2

∴ Factored DL + LL is

= 1.5 (0.296 + 0.4)

= 1.044 kN/m^2

= 1.044 × 1.4 = 1.46 kN/m, vertically downward

∴ Load normal to sheeting

= 1.46 cos θ = 1.46 cos 26.565

= 1.306 kN/m

Load in the direction parallel to sheeting

= 1.46 sin θ = 0.653 N/m.

Bending moments are:

$$M_z = 1.306 \times \frac{4.5^2}{8} = 3.306 \text{ kN-m}$$

$$M_y = 0.653 \times \frac{4.5^2}{8} = 1.653 \text{ kN-m}$$

Shear forces are:

$$F_z = 1.306 \times \frac{4.5}{2} = 2.939 \text{ kN}$$

$$F_y = 0.653 \times \frac{4.5}{2} = 1.469 \text{ kN}$$

Try ISMC 100 section.

$d = h_1 + 2r_1 = 64 + 2 \times 9 = 82$ mm $b = 50$ mm

$$Z_{pz} \text{ required} = \frac{3.306 \times 10^6}{250} \times 1.1 + 2.5 \times \frac{76}{50} \times \frac{1.653 \times 10^6}{250} \times 1.1$$

$$= 42.185\ 10^3 \text{ mm}$$

Z_{pz} of ISMC 100 is 43.825 × 10^3 mm^3. Hence adequate.

Check for shear:

$$V_{dz} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times ht_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 100 \times 4.7 = 61671 \text{ N} > F_z$$

$$V_{dy} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times (2b\,t_f) = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 2 \times 50 \times 7.5$$

$$= 94412 \text{ N} > F_y$$

Section is adequate to resist shear.

Design capacity of the section:

Section classification:

$$\frac{b}{t_f} = \frac{50}{7.5} = 6.67 < 9.4$$

$$\frac{d}{t_w} = \frac{82}{4.7} = 17.45 < 42$$

Hence it is plastic section.

$$\therefore \quad M_{dz} = \frac{Z_{pz} f_y}{\gamma_{mo}} = \frac{43.8 \times 10^3 \times 250}{1.1} = 9.955 \times 10^6 \text{ N-m}$$

$$= 9.955 \text{ kN-m}$$

$$M_{dy} = \frac{Z_{py} f_y}{\gamma_{mo}}$$

From steel table, $Z_{py} = 14208.25 \text{ mm}^3$

$$\therefore \quad M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}} = 14208.25 \times \frac{250}{1.1} = 3.229 \times 10^6 \text{ N-mm}$$

$$= 3.229 \text{ kN-m}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{3.306}{9.955} + \frac{1.653}{3.229} = 0.844 > 1$$

Check for wind condition:

At this stage live load is not to be considered since wind force is suction and hence critical load condition is when there is no live load.

For this condition of loading:

Factored DL = 1.5 × 0.4135 = 0.620 kN/m, vertically downward.

[*Note:* Though DL reduces net moment about z-axis, it increases moment about y-axis. Hence load factor should be taken as 1.5.]

Wind load $= 1.2 \text{ kN/m}^2 = 1.2 \times 1.4 = 1.68$ kN/m

$\therefore$ Factored wind load = 1.5 × 1.68 = 2.52 kN/m, suction wind load acts normal to sheeting.

$\therefore$ Load normal to sheeting $= -2.52 + 0.620 \cos 26.565°$

$$= -1.965 \text{ kN/m} = 1.965 \text{ kN/m outward}$$

Load parallel to sheeting $= 0.620 \sin 26.965°$

$$= 0.277 \text{ kN/m}$$

$$M_{zz} = 1.965 \times \frac{4.5^2}{8} = 4.974 \text{ kN-m}$$

$$\therefore \quad M_{yy} = 0.277 \times \frac{4.5^2}{8} = 0.701 \text{ kN-m.}$$

M_{dz} for laterally unsupported compression flange is to be found.

Effective length of simply supported beam with destabilizing loading (since load do not act through shear centre) = 1.2 L = 1.2 × 4500 = 5400 mm.

$$r_y = 19.2 \text{ mm}$$

$$\therefore \quad \lambda = \frac{kL}{r_y} = \frac{5400}{14.9} = 362.42 > 350$$

Hence revise the section (Ref. Table 5.1)

Try ISMC 125. For this $r_y = 19.2$ mm

$$\therefore \quad \lambda = \frac{5400}{19.2} = 281.25 \qquad \text{O.K.}$$

$$\frac{h}{t_f} = \frac{125}{8.1} = 15.43$$

From Table 14 of IS 800 (Table 7.3), f_{crb} is to be found by double linear interpolation. From the table

$h/t_f \rightarrow$ / $\lambda \downarrow$	14	15.43	16
280	126.9	X	111.9
281.25		O	
290	122.3	Y	107.8

$$\text{At } X, f_{crb} = 126.9 - \frac{1.43}{2}(126.9 - 111.9) = 116.18 \text{ N/mm}^2$$

$$\text{At } Y, f_{crb} = 122.3 - \frac{1.43}{2}(122.3 - 107.8) = 111.93 \text{ N/mm}^2$$

$$\therefore \quad \text{At } O, f_{crb} = 116.18 - \frac{1.25}{10}(116.18 - 111.93) = 115.65 \text{ N/mm}^2$$

From Table 13 (a) in IS 800 (Table 7.4)

$$f_{bd} = 77.3 + \frac{15.65}{50}(106.8 - 77.3) = 86.53 \text{ N/mm}^2$$

$$\therefore \quad M_{dz} = \beta_b Z_{pz} f_{bd} = 1 \times 77.2 \times 10^3 \times 86.53$$

$$= 66.8 \times 10^6 \text{ N-mm} = 66.8 \text{ kN-m}$$

$M_{dy} = 2.593$ kN-m (as found earlier).

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{4.974}{66.8} + \frac{0.701}{2.593} < 1.0$$

Hence the section ISMC 125 is adequate.

Check for Deflection

$I_z = 416.4 \times 10^4$ mm^4; $w = 1.306$ kN/m = 1.306 N/mm.

$$\therefore \quad \delta = \frac{5}{384} \times \frac{wL^4}{EI} = \frac{5}{384} \times \frac{1.306 \times 4500^4}{2 \times 10^5 \times 416.4 \times 10^4}$$

$$= 8.4 \text{ mm.}$$

$$\text{Permissible deflection} = \frac{L}{150} = \frac{4500}{150} = 30 \text{ mm}$$

Hence safe.

Provide ISMC 125 as purlin.

Example 7.10

Design an I-section purlin for an industrial building to support a galvanized corrugated iron sheet roof. Given:

Spacing of the trusses = 5.0 m

Spacing of purlins = 1.5 m.

Inclination of main rafter to horizontal = 30°

Weight of galvanized sheets taking into account laps and connecting bolts = 130 N/m^2

Imposed snow load = 1.5 kN/m^2

Wind load = 1.0 kN/m^2, suction.

Solution:

DL = 130 N/m^2, spacing of purlins = 1.5 m

$\therefore$ DL per metre run of purlin = 130 × 1.5 = 195 N/m

Imposed load = 1.5 kN/m^2

$\therefore$ Imposed load per metre run of purlin = 1.5 × 1.5 = 2.25 kN/m.

$\therefore$ Factored vertical load = 1.5 (DL + LL)

= 1.5 (0.195 + 2.25) = 3.668 kN/m

$\therefore$ Component of load normal to sheets

$w_z = 3.668 \cos 30° = 3.177$ kN/m

Component of load parallel to sheets

$w_y = 3.668 \sin 30° = 1.834$ kN/m

Design for DL + IL

$$M_z = \frac{w_z L^2}{8} = 3.177 \times \frac{5^2}{8} = 9.928 \text{ kN-m}$$

$$M_y = \frac{w_z L^2}{8} = 1.834 \times \frac{5^2}{8} = 5.731 \text{ kN-m}$$

$$F_z = 3.177 \times \frac{5}{2} = 7.943 \text{ kN}$$

$$F_y = 1.834 \times \frac{5}{2} = 4.585 \text{ kN}$$

$$Z_{pz} \text{ required} = \frac{M_z}{f_y} \gamma_{mo} + 2.5 \times \frac{d}{b} \times \frac{M_y \gamma_{mo}}{f_y}$$

Trying ISMB 150,

$$d = 150 - 2(18.05) = 113.9 \text{ mm}$$

$$b = 80 \text{ mm}$$

$$Z_{pz} \text{ required} = \frac{9.928 \times 10^6}{250} \times 1.1 + 2.5 \times \frac{113.9}{80} \times \frac{5.731 \times 10^6}{250} \times 1.1$$

$$= 133.438 \times 10^3 \text{ mm}^3$$

Z_{pz} of ISMB 150 is 110.476×10^3 mm^3. Hence not sufficient.
Try ISMB 175.

$$d = 175 - 2(20.25) = 134.5 \text{ mm}$$

$$b = 90 \text{ mm}$$

$$Z_{pz} \text{ required} = \frac{9.928 \times 10^6}{250} \times 1.1 + 2.5 \times \frac{134.5}{90} \times \frac{5.731 \times 10^6}{250} \times 1.1$$

$$= 137.894 \times 10^3 \text{ mm}^3.$$

Z_{pz} of ISMB 175 is 166.076×10^3 mm^3. Hence adequate.

Check for shear:

$$V_{dz} = \frac{f_y}{\sqrt{3}} \frac{1}{\gamma_{mo}} h t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 175 \times 5.5$$

$$= 126.295 \times 10^3 \text{ N} > F_z$$

$$V_{dy} = \frac{}{\sqrt{3}} \times \frac{1}{mo} \times (2bt_f) = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 2 \times 90 \times 8.6$$

$$= 203.122 \times 10^3 \text{ N} > F_y$$

Hence adequate.

Design capacity of the section:

Section classification:

$$f_y = 250 \therefore \epsilon = 1, \text{ length of outstand} = \frac{90}{2} = 45$$

$$\frac{b}{t_f} = \frac{45}{8.6} = 5.23 < 9.4$$

$$\frac{d}{t_w} = \frac{134.5}{5.5} = 24.45 < 84$$

Hence it belongs to plastic (category 1) section.

$$M_{dz} = \beta_b \frac{Z_{pz} f_y}{\gamma_{mo}} \times \frac{10 \times 166.076 \times 10^3 \times 250}{1.1} = 37.745 \times 10^6 \text{ N-mm}$$

$$= 37.745 \text{ kN-m.}$$

Now, $$\frac{1.2 Z_e f_y}{\gamma_{mo}} = 1.2 \times \frac{145.3 \times 10^3 \times 250}{1.1} = 39.627 \times 10^6 \text{ N-mm}$$

$$\therefore \quad M_{dz} = 37.745 \text{ kN-m}$$

$$M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}}$$

$$Z_{py} = 32094.02 \text{ mm}^3.$$

$$\therefore \quad M_{dy} = 32094.02 \times \frac{250}{1.1} = 7.294 \times 10^6 \text{ N-mm}$$

$$= 7.294 \text{ kN-m.}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{9.928}{37.745} + \frac{5.731}{7.294}$$

$$= 1.049 \text{ slightly more than 1.}$$

Hence adequate.

Check for wind condition:

Dead load component in *z*-direction reduces wind effect, but its component parallel to sheeting (*y*-direction) increases the stresses. Overall effect on purlin is to increase the stresses. Hence load factor 1.5 is to be used.

$\therefore$ Factored DL = $1.5 \times 0.195 = 0.292$ kN/m, vertically downward.

Factored wind load = 1.5 WL

$= 1.5 \times 1 = 1.5$ kN/m, suction.

It acts normal to sheeting.

$\therefore$ Load normal to sheeting $= -1.5 + 0.292 \times \cos 30° = -1.247$ kN/m

$= 1.247$ kN/m, outward.

Load parallel to sheeting $= 0.292 \sin 30°$

$= 0.146$ kN/m.

$$\therefore \quad M_z = 1.247 \times \frac{5^2}{8} = 3.897 \text{ kN-m}$$

$$M_y = 0.146 \times \frac{5^2}{8} = 0.456 \text{ kN-m}.$$

M_{dz} is to be found for the I-section with unsupported compression flange.

Since load acts through the shear centre the effective length of simply supported beam

$$L = 5 \text{ m} = 5000 \text{ mm}$$

$$r_y = 18.6 \text{ mm}$$

$$\lambda = \frac{L}{r_y} = \frac{5000}{18.6} = 268.8$$

$$\frac{h}{t_f} = \frac{175}{8.6} = 20.35$$

From Table 14 of IS 800 (Table 7.3 in this book), f_{crb} is to be found by double linear interpolation.

$\frac{h}{t_f} \rightarrow$ / $\gamma \downarrow$	20	20.35	25
260	98.8	X	81.3
268.8		O	
270	94.7	Y	77.9

$$\text{At } X,\ f_{crb} = 98.8 - \frac{0.35}{5}(98.8 - 81.3) = 97.58 \text{ N/mm}^2$$

$$\text{At } Y,\ f_{crb} = 94.7 - \frac{0.35}{5}(94.7 - 77.9) = 93.52 \text{ N/mm}^2$$

$$\therefore \quad \text{At } O,\ f_{crb} = 97.58 - \frac{8.8}{10}(97.58 - 93.52) = 94 \text{ N/mm}^2$$

From Table 13 (a) in IS 800 (Table 7.4 in this book)

$$f_{bd} = 70.5 + \frac{4}{10}(77.3 - 70.5) = 73.22 \text{ N/mm}^2$$

Since it is plastic section,

$$M_{dz} = 1 \times Z_{pz} \times f_{bd} = 166.076 \times 10^3 \times 73.22 = 12.160 \times 10^6 \text{ N-mm}$$

$$= 12.160 \text{ kN-m.}$$

$$M_{dy} = 7.294 \text{ kN-m as found earlier.}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{3.897}{12.160} + \frac{0.456}{7.294} < 1$$

Hence adequate.

Check for Deflection:

$$I_z = 1272 \times 10^4 \text{ mm}^4.$$

$$w = 3.668 \text{ kN/m. as found for (DL+LL) case}$$

$$= 3.668 \text{ N/mm}$$

$$\therefore \quad \delta = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384} \times \frac{3.668 \times 5000^4}{2 \times 10^5 \times 1272 \times 10^4}$$

$$= 11.73 \text{ mm.}$$

Permissible deflection $= \dfrac{L}{150} = \dfrac{5000}{150} = 33.33 \text{ mm} > 11.73 \text{ mm}$, hence safe,

$\therefore$ Provide ISMB 175 as purlin.

7.15 SIMPLIFIED METHOD FOR THE DESIGN OF ANGLE PURLINS

As per IS 800-2007, angle purlins should be designed for biaxial bending. However IS 800-1984 and present British code permit design of angle purlins by assuming that load normal to sheeting is resisted by purlin and load parallel to sheeting is resisted by sheeting, provided the following conditions are fullfilled:

(a) Roof slope is less than 30°.
(b) Width of angle leg perpendicular to sheeting $\geq \dfrac{L}{45}$.
(c) Width of angle leg parallel to sheeting $\geq \dfrac{L}{60}$.

In the above situations, bending moment about z-z axis should be taken as $\dfrac{W_z L}{10}$, where W_z is total load in the direction normal to sheeting and L is the spacing of trusses.

Thus the problem is treated as uni-axial bending. The specifications about minimum leg sizes are based on limiting deflections. Hence in this method of design there is no need to check for the deflection. Unfortunately this simplified method is not included in IS 800-2007.

Example 7.11

Design angle purlin for the following data by simplified method:

Spacing of trusses = 3.5 m.

Spacing of purlins = 1.6 m

Weight of A.C. sheets including laps and fixtures = 0.205 kN/m^2

Live load = 0.6 kN/m^2

Wind load = 1 kN/m^2, suction

Inclination of main rafter of truss = 21°.

Solution:

Trial section:

$$\text{Minimum depth} = \frac{L}{45} = \frac{3500}{45} = 78 \text{ mm}$$

$$\text{Minimum width} = \frac{L}{60} = \frac{3500}{60} = 58.3 \text{ mm}$$

Let us try ISA 9060, 6 mm thick.

Dead Load:

Weight of AC sheets with overlap and fixtures = 0.205 kN/m^2

$$= 1.6 \times 0.205 = 0.328 \text{ kN/m}$$

Live load = 0.6 × 1.6 = 0.96 kN/m

∴ Total vertical downward load = 0.328 + 0.96

$$= 1.288 \text{ kN/m}$$

Factored (LL + DL) normal to sheeting = 1.5 × (1.288) cos 21°

$$= 1.804 \text{ kN/m}$$

Factored (DL + WL) normal to sheeting = 1.5 (0.328 cos 21° – 1.0) = –1.04 kN/m = 1.04 kN/m outward.

∴ Live load + Dead load is critical.

$$M_z = \frac{WL^2}{10} = \frac{1.804 \times 3.5 \times 3.5}{10} = 2.210 \text{ kN-m}$$

For ISA 9060, 6 mm thick,

$$\frac{b}{t_f} = \frac{60}{6} = 10.0 \text{ between 9.4 and 10.5}$$

$$\frac{b}{t_w} = \frac{90}{6} = 15 \text{ between 10.5 and 15.7}$$

Hence it belongs to class 3 (semi compact) section.

For such section,

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{mo}} = \frac{Z_e f_y}{\gamma_{mo}}$$

For ISA 10075, 6 mm

$$Z_e = 11.5 \times 10^3 \text{ mm}^3.$$

$$\therefore \quad M_d = \frac{11.5 \times 10^3 \times 250}{1.1} = 2.875 \times 10^6 \text{ N-mm}$$

$$= 2.875 \text{ kN-m} > M_z$$

Hence ISA 10075, 6 mm is suitable as purlin.

7.16 DESIGN OF GRILLAGE BEAMS

High rise buildings are built with steel columns encased in concrete. Such columns carry very heavy loads and they need special foundation to spread the load to soil. The foundation consists of one tier or more tires of I-section beams. Such foundation is called grillage foundation. Figure 7.14 shows a typical two tier grillage foundation.

The grillage beams are unpainted and encased in concrete with minimum cover of 100 mm beyond the edges of steel beams. A minimum clear space of 75 mm should be maintained between the flanges of adjacent grillage beams so that concreting can be done properly. To maintain spacing tie rods are used.

The column load is transferred to top tier of grillage beams through a base plate.

Let the length of grillage beam be L and the length of plate in this direction be 'a' as shown in Fig. 7.15.

Maximum moment occurs at centre of beam

If a beam carries a total load P, pressure under beam is $\frac{P}{L}$ and on it is $\frac{P}{a}$.

$$M = \frac{P}{L} \times \frac{L}{2} \times \frac{L}{4} - \frac{P}{a} \cdot \frac{a}{2} \cdot \frac{a}{4}$$

$$= \frac{P(L-a)}{8}$$

Maximum shear occurs at distance 'a' from the centre of the beam and its value is

$$F = \frac{P}{L}\left(\frac{L}{2} - \frac{a}{2}\right) = \frac{P(L-a)}{2L}$$

The grillage beams should be designed for the above moment and shear. It should be checked for web crippling. The design procedure is illustrated with the example below:

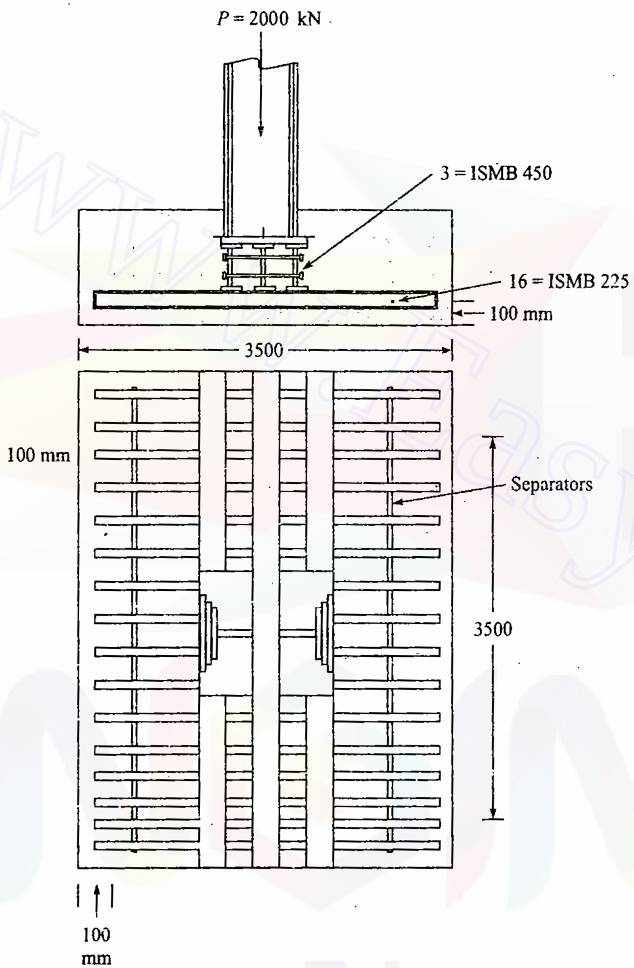


Figure 7.14 Steel grillage foundation

Example 7.12

Design a two tier grillage foundation for a column carrying 2000 kN load. The size of base plate is 800 × 800 mm. Safe bearing capacity of the soil is 200 kN/m^2.

Solution:

Load from column = 2000 kN

Self weight of footing, say = 200 kN

∴ Total load on soil = 2200 kN

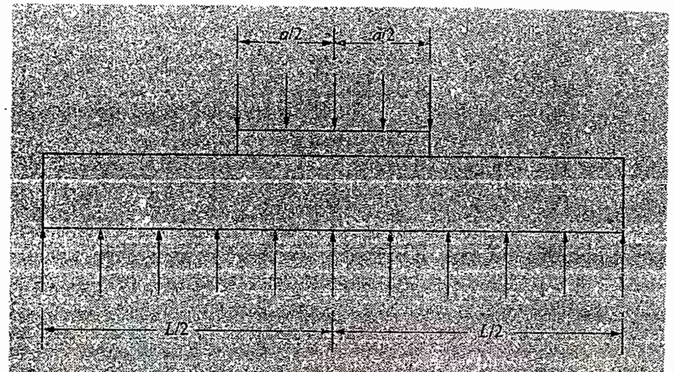


Figure 7.15

SBC of soil = 200 kN/m^2

$$\therefore \quad \text{Area of footing required} = \frac{2200}{200} = 11\ \text{m}^2$$

Provide 3.5 m × 3.5 m footing.

Design of beam in upper tier:

$L = 3.5$ m, $\quad a = 0.8$ m

Factored load = 2000 × 1.5 = 3000 kN

$$\therefore \quad \text{Factored } M = \frac{3000\,(3.5 - 0.8)}{8} = 1012.5 \text{ kN-m}$$

$$\text{Factored shear force } V = 3000\frac{(L-a)}{2L}$$

$$= \frac{3000(3.5-0.8)}{2 \times 3.5}$$

$$= 1157.142 \text{ kN}$$

Section modulus required is given by

$$Z_{pz} = \frac{M}{f_y}\,\gamma_{mo} = \frac{1012.5 \times 10^6}{250} \times 1.1 = 4455 \times 10^3 \text{ mm}^3$$

Providing 3 beams,

$$Z_p \text{ of each beam} = \frac{4455 \times 10^3}{3} = 1485 \times 10^3 \text{ mm}^3$$

Use ISMB 450 which has $Z_p = 1553.4 \times 10^3$ mm^3.

Check for clear spacing:

Width of flange of ISMB 450 = 150 mm

$$\text{Clear spacing} = \frac{800 - 3 \times 150}{3} = 116.67 \text{ mm} > 75 \text{ mm}$$

Hence adequate.

Section classification: Outstand of leg $b = \frac{150}{2} = 75$

$$\frac{b}{t_f} = \frac{75}{17.4} = 4.31 < 10.5 \quad \text{and} \quad \frac{d}{t_w} = \frac{450 - 2(17.4 + 15)}{9.2} < 84$$

$\therefore$ It belongs to class 1 (plastic) category.

$$\therefore \quad M_d = \frac{1 \times Z_p f_y}{\gamma_{mo}} = \frac{1 \times 1554.347 \times 10^3 \times 250}{1.1} = 353.033 \times 10^6 \text{ N-mm}$$

$$= 353.033 \text{ kN-m}$$

Moment to be resisted by each beam = $\frac{1012.5}{3}$ = 337.5 kN-m

$\therefore$ $M_d > 337.5$. Hence the section is adequate.

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 450 \times 9.4$$

$$= 555044 \text{ N} = 555.044 \text{ kN}$$

Shear to be resisted by each beam

$$V = \frac{1157.142}{3} = 385.714 \text{ kN} < V_d$$

Hence safe.

Design capacity:

$0.6\, v_d = 0.6 \times 555.044 = 333$ kN $< V$

$\therefore$ Bending strength should be checked for high shear condition (clause 9.2.2).

M_{fd} = plastic design strength of the area of the cross-section, excluding the shear area, considering partial safety factor γ_{mo}.

= plastic design strength of flanges only.

Referring to Fig. 7.16, we get

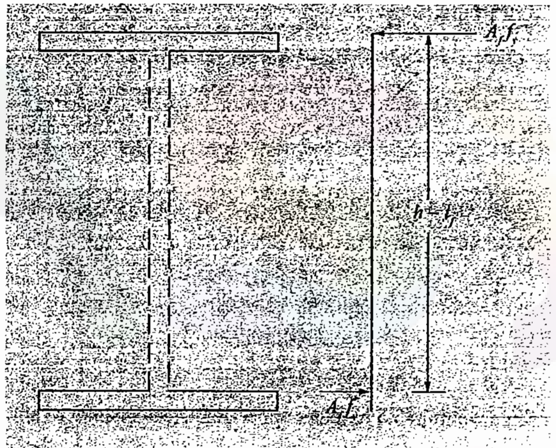


Figure 7.16

$$M_{fd} = f_y A_f (h - t_f) \times \frac{1}{\gamma_{mo}}$$

$$= 250 \times 150 \times 17.4 (450 - 17.4) \times \frac{1}{1.1}$$

$$= 256.61 \times 10^6 \text{ N-mm} = 256.61 \text{ kN-m}$$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2 \times 385.714}{555.044} - 1\right)^2 = 0.152$$

$$\therefore \quad M_{dv} = M_d - \beta (M_d - M_{fd}) \leq 1.2\, Z_e f_y \times \frac{l}{\gamma_{mo}}$$

$$= 353.045 - 0.152 \; (353.045 - 256.61) \leq 1.2 \times 1350.7 \times 10^3 \times 250 \times \frac{1}{1.1}$$

$$= 338.387 \text{ kN-m} < 368.37 \text{ kN-m}$$

$\therefore \quad M_{dv} = 338.387$ kN-m

$\therefore \quad M_{dv} = 338.387 > M$ of each section

Hence safe.

Check for web crippling:

Now, $b_1 = 800$ mm, $t_w = 9.4$ mm

$t_f = 17.4$ mm and $r_1 = 15$ mm

Since dispersion takes place at to edges, $n_2 = 2 \times 2.5\ (h_2) = 2 \times 2.5 \times 35.4$.

$$F_w = (b_1 + n_2)\, t_w f_y \frac{1}{\gamma_{mo}} = [800 + 2 \times 2.5\,(35.4)]\ \frac{9.4 \times 250}{1.1}$$

$$= 2087.227 \times 10^3 \text{ N} > \frac{3000}{3} \text{ kN,} \qquad \textbf{Hence O.K.}$$

Design of beams in lower tier:

$$M = \frac{P}{8}(L-a) = 3000\,\frac{3.5-0.8}{8} = 1012.5 \text{ kN-m}$$

$$V = 3000\frac{(L-a)}{2L} = \frac{3000(3.5-0.8)}{2 \times 3.5} = 1157.142 \text{ kN}$$

Beams are to be placed in a width of 3.5 m under upper tier. Clear spacing required is 75 mm. Assuming 16 beams in this tier.

Hence clear spacing is sufficient.

$$\text{M for each beam} = \frac{1012.5}{16} = 63.281 \text{ kN-m}$$

$$\text{V for each beam} = \frac{1157.142}{16} = 72.321 \text{ kN}$$

$$Z_p \text{ required} = \frac{M}{f_y}\gamma_{mo} = \frac{63.281 \times 10^6}{250} \times 1.1 = 278436 \text{ mm}^3$$

Provide ISMB 225. Hence Z_{pz} provided $= 348.271 \times 10^3$ mm^3. b = 110 mm.

$$\therefore \text{Clear spacing} = \frac{3500 - 16 \times 110}{15} = 116 \text{ mm} > 75 \text{ mm.} \qquad \textbf{Hence O.K.}$$

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 225 \times 6.5 = 191.903 \times 10^3 \text{ N}$$

$$= 191.903 \text{ kN} > V$$

$\therefore$ Adequate and also $0.6\, V_d > V$.

Check for moment:

$$\text{Outstanding leg} = \frac{110}{2} = 55$$

$$\frac{b}{t_f} = \frac{55}{11.8} = 4.66 < 9.4$$

$$\frac{d}{t_w} = \frac{225 - 2(11.8 + 12)}{6.5} = 27.29 < 84$$

Hence it is plastic (class 1) section.

$$\therefore \quad M_d = 1 \times Z_p \times f_y \times \frac{1}{\gamma_{mo}} = 1 \times 348.271 \times 10^3 \times 250 \times \frac{1}{1.1}$$

$$= 79.153 \times 10^6 \text{ N-mm} > 63.281 \text{ kN-mm}$$

Hence adequate.

Check for web crippling:

$b_1 = 800$ mm, $t_w = 6.5$ mm

$t_f = 11.8$ mm and $r_1 = 12$ mm

Since dispersion takes place at two edges,

$$h_2 = 25.85$$

$$\therefore \quad F_w = [800 + 2 \times 2.5\,(25.85)] \times 6.5 \times \frac{250}{1.1}$$

$$= 1372.756 \times 10^3 > \frac{3000}{16} \times 10^3 \text{ N}$$

Hence adequate.

Thus the grillage foundation consists of 3 ISMB 450 in upper tier and 16 ISMB 225 in lower tier. Each beam is of length 3.5 m. Cover plate provided is of size 800 mm × 800 mm.

* * *

Questions

1. Distinguish between laterally restrained and unrestrained beams.
2. Determine the plastic modules of section of I-section and channel section shown in Fig. 7.17 about z-z axis. If the shear area of the section is neglected, what percentage of plastic modulus of the section is reduced.

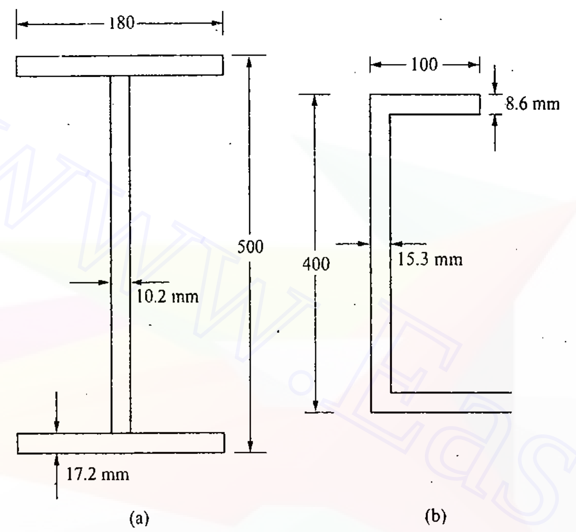


Figure 7.17

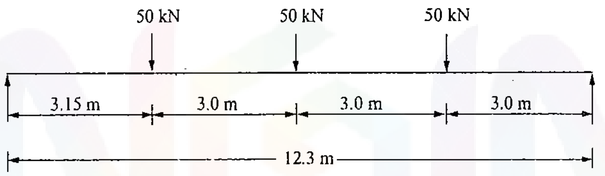


Figure 7.18

3. Design the main floor beam shown in Fig. 7.18. The beam is laterally restrained. Check for shear and deflection.
4. A hall measuring 15 m × 6 m consists of beams spaced at 3 m c/c. R.C.C. slab of 120 mm is cast over the beam. The imposed load is 4 kN/m^2. The beam is supported on 300 mm wall. Design one intermediate beam and check the design for deflection, web buckling and web crippling.
5. Design a simply supported beam of 9 m effective span carrying a load of 40 kN/m. The depth of the beam should not exceed 450 mm. The compression flange of the beam is laterally supported. Assume stiff end bearing is 80 mm.
6. Design a beam of 5 m effective span, carrying a uniform load of 20 kN/m, if the compression flange is laterally unsupported. Assume f_y = 250 MPa.
7. Design a channel section purlin for the following data:

Spacing of trusses = 4.0 m
Spacing of purlins = 1.8 m
Weight of sheets = 100 N/m^2
Weight of purlin = 100 N/m
Live load = 0.5 kN/m^2
Wind load = 1.5 kN/m^2, suction
Inclination of main rafter = 20°.

8. Design an I-section purlin for an industrial building to support a galvanized corrugated iron sheet. Given:
   Spacing of the trusses = 6 m
   Inclination of main rafter = 30°
   Spacing of purlins = 1.5 m
   Weight of corrugated sheeting = 130 N/m^2
   Live load = 0.6 kN/m^2
   Wind load = 1.8 kN/m^2, suction.
   Yield stress of steel = 250 MPa.
9. Design angle iron purlin for the following data assuming bending in the direction of sheeting is resisted by sheets:
   Spacing of trusses = 4.5 m
   Spacing of purlins = 1.5 m
   Weight of A.C. Sheets = 0.171 kN/m^2
   Live load = 0.56 kN/m^2
   Wind load = 1.2 kN/m^2, suction.
   Indication of main rafter to horizontal = 25°.
10. Design a two tier grillage foundations for a column carrying 1600 kN load. The size of the base plate is 700 × 700 mm. Safe bearing capacity of the soil is 200 kN/m^2.

8. Design an I-section purlin for an industrial building to support a galvanized corrugated iron sheet. Given:
   Spacing of the trusses = 6 m
   Inclination of main rafter = 30°
   Spacing of purlins = 1.5 m
   Weight of corrugated sheeting = 130 N/m^2
   Live load = 0.6 kN/m^2
   Wind load = 1.8 kN/m^2, suction.
   Yield stress of steel = 250 MPa.
9. Design angle iron purlin for the following data assuming bending in the direction of sheeting is resisted by sheets:
   Spacing of trusses = 4.5 m
   Spacing of purlins = 1.5 m
   Weight of A.C. Sheets = 0.171 kN/m^2
   Live load = 0.56 kN/m^2
   Wind load = 1.2 kN/m^2, suction.
   Indication of main rafter to horizontal = 25°.
10. Design a two tier grillage foundations for a column carrying 1600 kN load. The size of the base plate is 700 × 700 mm. Safe bearing capacity of the soil is 200 kN/m^2.

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8

DESIGN OF BOLTED BEAM CONNECTIONS

Beams are connected to main beams or to the columns. Design of these connections are more important since failure of connection is more catastrophic than the failure of beam section. In this chapter different types of beam connections are explained and designed.

8.1 TYPES OF BEAM CONNECTIONS

According to the degree of rotation permitted in the connection beam connections are classified as:

1. *Simple Connection or Flexible Connection*: In this type of connections no restraint is imposed for rotation. In other words, the connection is designed to transfer the end shear only.
2. *Moment Resistant or Rigid Connection*: In this the joint is designed to resist end shear as well as moment. Such supports may be treated as fixed ends, since they do not permit any rotation at the ends.
3. *Semi Rigid Connection*: In this type rotation of end is partially restrained. In other words, the connections are designed to transfer shear and part of fixed end moments.

Simple beam connections may be further classified as:

(i) Framed connection
(ii) Unstiffened seated connection
(iii) Stiffened seated connection.

(i) *Framed Connections*: When end shear to be transferred is less, it is possible to connect the beam to main beam or to the column using cleat angles as shown in Figs. 8.1(a) and (b). If the flanges of beam to be connected are at the same level, the flanges of connecting beam are cut as shown in Fig. 8.2. This will not pose any structural problem, since at the end of simply supported beams moment is zero and shear strength depends mainly on the strength of web.

(ii) *Unstiffened Seated Connection*: When shear force is larger the depth of cleat angle required for framed connection may be more than that can be provided in the available space. In such cases seat angles are connected to the column over which beam rests. At top cleat angles are provided to prevent

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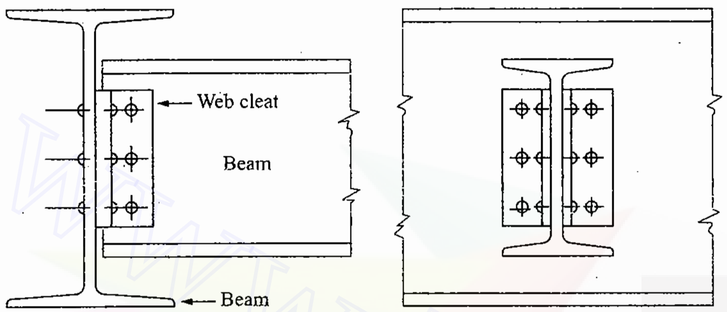
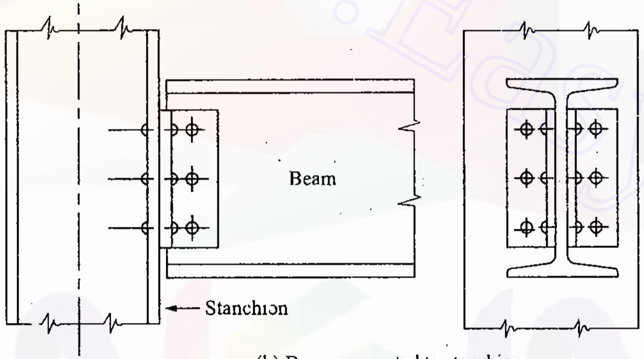


(a) Beam connected to beam



(b) Beam connected to stanchion

Figure 8.1 Framed connection.

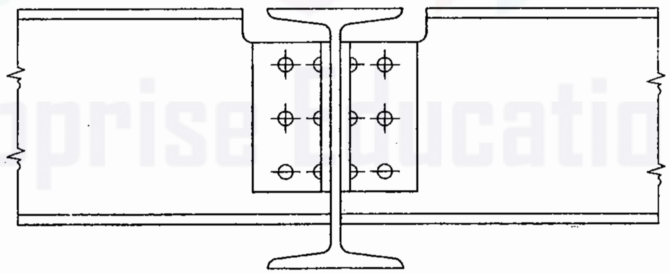


Figure 8.2 Framed connnection if flanges are at the same level.

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the lateral displacement of the beam after positioning it over seat angle. Figure 8.3 shows a typical such connection.

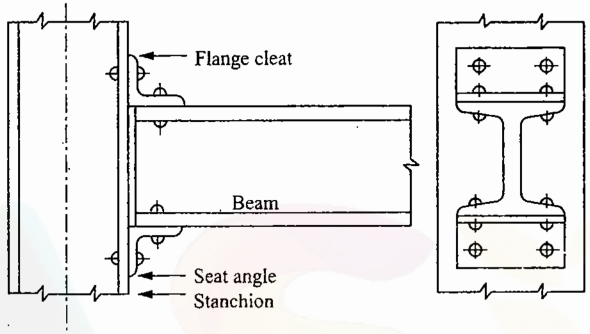

Figure 8.3 Unstiffened seated connection.

(iii) *Stiffened Seated Connection*: If shear force to be transferred in the beam is still large, the seat angle may fail. To strengthen it, a stiffener angle may be provided as shown in Fig. 8.4. Such connections are known as stiffened seated connection.

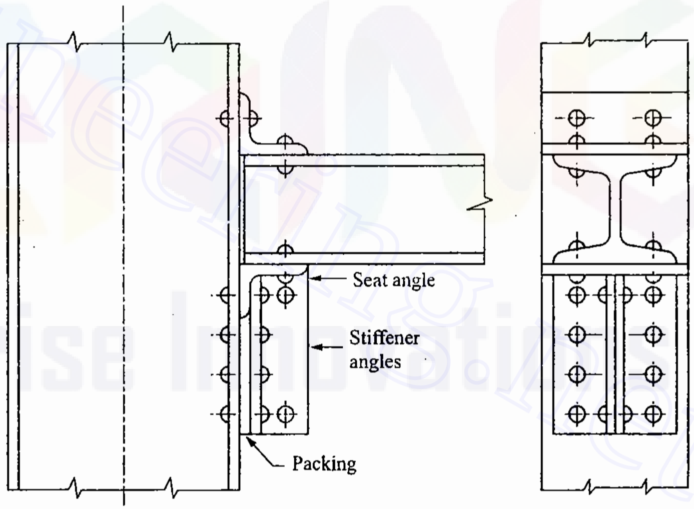

Figure 8.4 Stiffened seated connection.

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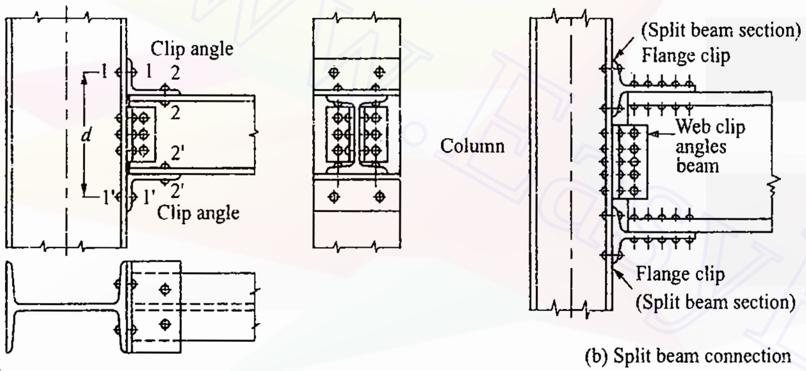
The moment resistant connections may be further classified as

(i) Clip-angle or split beam connection
(ii) Bracket connection

(i) *Clip angle connection:* This type of connection can be used at the end, if the moment to be transferred in the end is small. Figure 8.5 shows a typical clip angle connection.



(a) Clip angle connection

(b) Split beam connection

Figure 8.5

(ii) *Bracket connection:* If the moment to be transferred through the connection is large such connections are used Fig. 8.6 shows a typical bracket connection.

8.2 DESIGN OF FRAMED CONNECTIONS USING BOLTS

The bolts connecting web of beam to cleat angles are in double shear. The number of bolts required to transfer end shear should be determined. They may be accommodated in a single or double row depending upon the availability of depth of web.

The bolts connecting cleat angle to the main beam or columns are under single shear. They are to be designed to resist end shear.

The size of cleat angle depends on the number of rows of bolts provided. Minimum edge distances and spacing decide the size. Thickness of cleat angle should be so selected that the strength of cleat angle per pitch width is more than the strength of a bolt. Example 8.1 illustrates such design.

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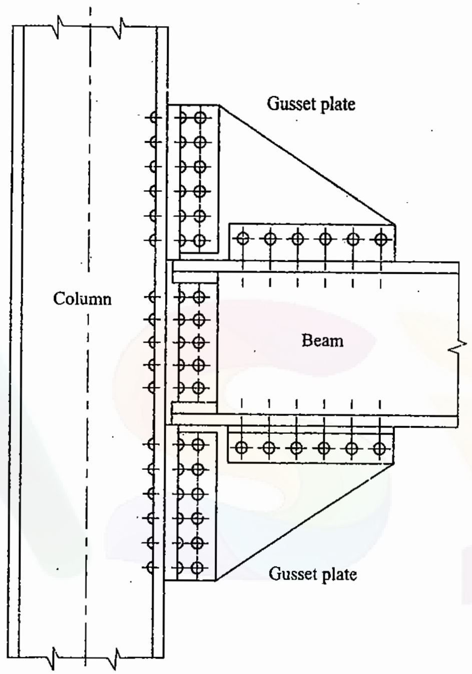

Figure 8.6 Bracket connection.

Example 8.1

An ISLB 300 carrying udl of 50 kN/m has effective span of 8 m. This is to be connected to the web of girder ISMB 450. Design the framed connection using 20 mm black bolts.

Solution:

1. *Connection of cleat angle with the web of secondary beam:*

Strength of M20 bolts in double shear

$$= \frac{f_{ub}}{\sqrt{3}} \times \frac{1}{\gamma_{mb}} \times (1 + 0.78)\frac{\pi}{4} \times 20^2$$

$$= \frac{400}{\sqrt{3}} \times \frac{1}{1.25} \times 1.78 \times \frac{\pi}{4} \times 20^2$$

$$= 103.314 \times 10^3 \text{ N} = 103.314 \text{ kN}.$$

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Strength in bearing over web of ISLB 300:

Providing an edge distance $e = 40$ mm and pitch $p = 60$ mm, we find

K_b is the minimum of $\dfrac{40}{3\times22}$, $\dfrac{60}{3\times22} - 0.25$, $\dfrac{400}{410}$, 1.0.

$\therefore \quad K_b = 0.606, \qquad \text{Now } t = t_w = 6.7 \text{ mm}$

Strength in bearing $= 2.5 K_b\, dt\, f_u \times \dfrac{1}{\gamma_{mb}}$

$$= 2.5\times0.606\times20\times6.7\times410\times\frac{1}{1.25}$$

$$= 66587 \text{ N} = 66.587 \text{ kN}$$

$\therefore$ Bolt value = 66.587 kN

End reaction $= 50\times\dfrac{8}{2} = 200$ kN

$\therefore$ Factored reaction $V = 1.5\times200 = 300$ kN

$\therefore$ No. of bolts required $= \dfrac{300}{66.587} = 4.500$

Provide 6 bolts in two rows.

2. *Connection of angle with web of girder:*

Thickness of web of girder (ISMB 450) = 9.4 mm

Strength of bolt in single shear

$$= \frac{f_{ub}}{\sqrt{3}}\times\frac{1}{1.25}\times0.78\times\frac{\pi}{4}d^2$$

$$= \frac{400}{\sqrt{3}}\times\frac{1}{1.25}\times0.78\times\frac{\pi}{4}\times20^2 = 45272 \text{ N}$$

$$= 45.272 \text{ kN}$$

Strength in bearing is more than it.

$\therefore$ Bolt value = 45.272 kN.

$\therefore$ No. of bolts required $= \dfrac{300}{45.272} = 6.6$

Provide 4 bolts in each angle at 50 mm spacing [Note even with this spacing strength of bolt in bearing is more than strength in single shear].

Design of cleat angle:

To keep the bearing strength on cleat angle greater than strength in single shear, thickness of cleat angle is given by

$$2.5\, K_b\, dt\, f_u \times \frac{1}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}}\times\frac{1}{\gamma_{mb}}\times0.78\times\frac{\pi}{4}\times d^2$$

$$2.5\times0.606\times20\times t\times410\times\frac{1}{1.25} = \frac{400}{\sqrt{3}}\times\frac{1}{1.25}\times0.78\times\frac{\pi}{4}\times20^2$$

$$t = 4.56 \text{ mm.}$$

Use 6 mm thick angle.

Provide ISA 15075, 6 mm angle with 150 mm leg on secondary beam so that, two rows of bolts may be provided in it.

Depth of angle required on secondary beam = 40 + 60 + 60 + 40 = 200 mm

Depth of angle required on main beam (girder) = 40 + 50 + 50 + 50 + 40 = 230 mm.

Provide 230 mm long cleat angle as shown in Fig. 8.7.

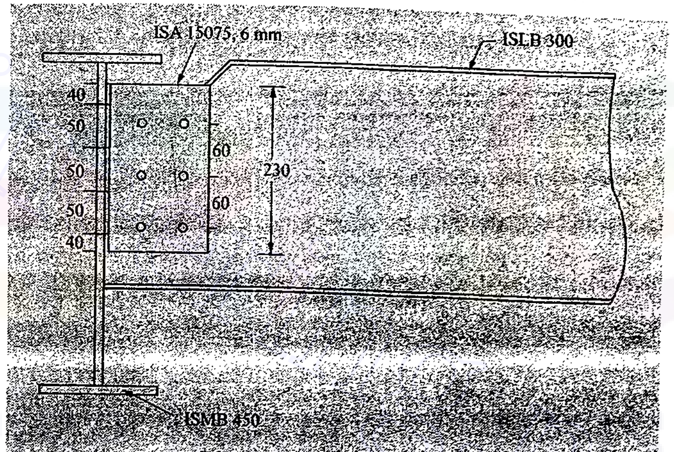

Figure 8.7

8.3 DESIGN OF UNSTIFFENED SEATED CONNECTIONS

In this type of connection, the seat angle over which beam rests is bolted to the column in shop and cleat angles are bolted in the field. The following procedure may be followed in the design:

1. The end reaction F is considered uniformly distributed on the outstanding leg of seat angle over a length 'b'. The length 'b' is such that web crippling of beam does not occur. This length may be calculated as,

$$b = B - \sqrt{3}\,h_2 \text{ subject to a minimum of } \frac{B}{2}$$

where $B = \dfrac{F}{f_p t_w}$

in which, F = shear force at working load

f_p = permissible bearing stress in the member

$= 0.75 f_y = 0.75 \times 250 = 187.5$ N/mm² for standard steel

h_2 = Depth at root of fillet from extreme fibre of flange

2. Select a trial section for seat angle.
3. Provide a clearance of 10 mm between the beam and the column. Compute the distance of end reaction from the critical section which is at the end of root of fillet (h_2) from extreme fibre of flange [Ref. Fig. 8.8]. Calculate the bending moment M at the critical section xx.

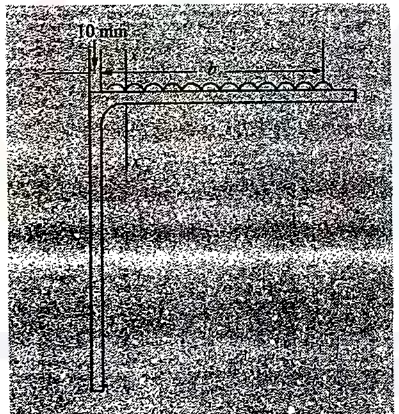

Figure 8.8

4. Assume the length of the seating angle to be equal to the width of the flange of the beam. Compute the moment of resistance of the seating angle, which should be greater than bending moment.

5. The seating angle is connected to the stanchion in the workshop. Compute the number of bolts required.
6. Connect a cleat angle at the top by two bolts in either leg. The diameter of bolts should be equal to the diameter of the bolts used to connect the seating angle.

Example 8.2

An ISMB 450 is to be connected to the flange of a column ISHB 300 @ 618 N/m. The end reaction transmitted by the beam is 120 kN. Design an unstiffened seated connection. Use M20 black bolts.

Solution:

Shear force at working load = 120 kN.

For ISMB 450, $t_w = 9.4$ mm, $t_f = 17.4$ mm

$h_2 = 35.4$ mm

$$B = \frac{F}{f_p t_w} = \frac{120 \times 10^3}{187.5 \times 9.4} = 68.09 \text{ mm}$$

$\therefore\ b = B - \sqrt{3}\,h_2$, subject to a minimum of $\dfrac{B}{2}$

$= 68.09 - \sqrt{3} \times 35.4$, subject to a minimum of 34.05 mm

$= 6.78$, subject to a minimum of 34.05 mm

$\therefore\ b = 34.05$ mm [Ref. Fig. 8.9]

Try ISA 150115, 12 mm.

$r_1 = 11$ mm.

$\therefore$ Critical section x-x is at

$= 12 + 11 = 23$ mm, from the back of the angle.

Centre of gravity of load is at

$$= 10 + \frac{34.05}{2} = 27.03 \text{ mm}$$

Factored force $= 1.5 \times 120$ kN.

$\therefore$ Factored moment $= 1.5 \times 120\,(27.03 - 23) = 724.5$ kN-mm

$= 724.5 \times 10^3$ N-mm

Length of seating angle = Width of flange of beam

= 150 mm

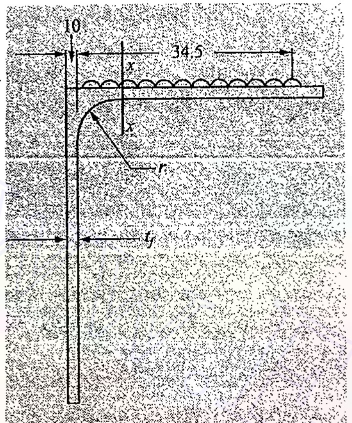

Figure 8.9

Z_p of rectangular section of width 'b', thickness 't': [Ref. Fig. 8.10]

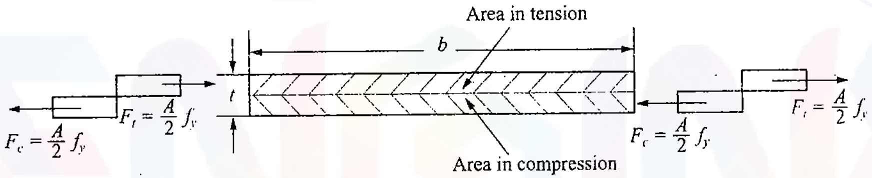

Figure 8.10

$$M_p = \frac{1}{2} b \times t \times f_y \times \frac{t}{2} = \frac{bt^2}{4} f_y$$

$$\therefore \quad Z_p = \frac{bt^2}{4}$$

$$\therefore \quad M_d = \frac{f_y Z_p}{\gamma_{mp}} = \frac{250}{1.25} \times \frac{bt^2}{4} = \frac{250}{1.25} \times \frac{150 t^2}{4} = 7500 t^2$$

Equating it to applied moment, we get

$$7500\, t^2 = 724.5 \times 10^3$$

$$\therefore \quad t = 10.099 \text{ mm}$$

Hence try 10 mm thick ISA.

Bolts connecting seat angle to column: M20 shop black bolts are used.

$$\text{Strength in single shear } = 0.78 \times \frac{\pi}{4} d^2 \frac{f_{ub}}{\sqrt{3}} \times \frac{1}{1.25}$$

$$= 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25}$$

$$= 45.272 \times 10^3 \text{ N}$$

Strength in bearing is more than it if minimum edge distance and pitch are maintained.

$$\therefore \quad \text{Bolt value} = 45.272 \times 10^3 \text{ N}$$

$$\text{Number of bolts required } = \frac{1.5 \times 120}{45.272} = 3.97$$

Provide 4 M20 bolts in two rows.

Hence use ISA 150115, 10 mm thick angle so that 2 rows bolts can be provided (Fig. 8.11). Connect top cleat angle ISA 10075, 10 mm with 2 field bolts of M20 in either leg of angle.

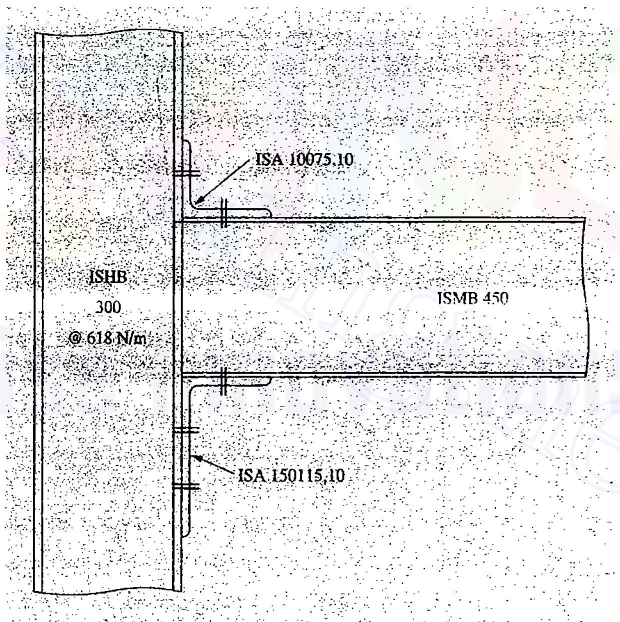

Figure 8.11

8.4 DESIGN OF STIFFENED SEATED CONNECTION

If end reaction to be transferred is more, the thickness of seat angle required will be larger than the available thickness or the number of bolts required in the vertical leg may be too many to be accommodated in the available width. Hence it is not possible to provide unstiffened seated connection. In such situations the seat angle needs stiffening from below by one or two angles.

In this case the bearing length 'b' is measured from the end of the stiffening leg.

The stiffening leg should have sufficient bearing area. Its thickness may be kept not less than the web thickness of the supported beam. To avoid local buckling, the ratio of outstanding leg length to its thickness should be less than 16.

The bolts in the column should be checked for direct shear and stress arising due to moment as explained in problem 8.3.

Example 8.3

An ISMB 500 beam transmits an end reaction of 250 kN to the web of a column ISHB 300 @ 577 N/m. Design and sketch a stiffened seated connection. Use M24 black bolts.

Solution:

Bearing length required from crippling consideration:

$$B = \frac{F}{f_p t_w} = \frac{250\times10^3}{187.5\times10.2} = 130.72 \text{ mm}$$

$$\therefore \quad b = B - \sqrt{3}\ h_2 = B - \sqrt{3}(37.96) = 130.72 - \sqrt{3}(37.96)$$

$$= 64.97 \text{ mm}$$

$$\frac{B}{2} = \frac{130.72}{2} = 65.36 \text{ mm}$$

$$\therefore \quad b = 64.97 \text{ mm.}$$

Provide a clearance of 10 mm.

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∴ Length of outstanding leg of seat angle = 64.97 + 10 = 74.97 mm

∴ Use ISA 9060, 10 mm with 90 mm leg as horizontal outstanding leg = 90 – 10 = 80 mm.

$$\frac{\text{Outstanding leg}}{\text{Thickness}} = \frac{80}{10} = 8 < 16$$

O.K.

Bearing area of stiffening angle required $= \dfrac{250\times10^3}{187.5} = 1333.3 \text{ mm}^2$

Provide 2 ISA 8080, 10 mm as stiffeners.

Bearing area provided = 2 (80 – 10) × 10 = 1400 mm^2 >1333.3 mm^2

Hence adequate.

Design of Bolts Connecting Stiffening Angles to Column

The distance of end reaction of the beam from end of stiffener angle

$$= \frac{b}{2} = \frac{64.97}{2} = 32.49 \text{ mm}$$

The distance of end reaction from the face of web of column = e = 90 + 10 – 32.49 = 67.51 mm

Bending moment about the face of the web

$$= 67.51 \times 250 = 16.878 \times 10^3 \text{ kN-mm} = 16.878 \text{ kN-m}$$

Factored Moment $M = 1.5 \times 16.878 = 25.316$ kN-m

Factored Shear $V = 1.5 \times 250 = 375$ kN

Strength of M24 bolts, shop driven in single shear

$$V_{db} = 0.78\times\frac{\pi}{4}\times24^2\times\frac{400}{\sqrt{3}}\times\frac{1}{1.25}$$

$$= 65192 \text{ N.}$$

Taking e = 45 mm, pitch p = 70 mm.

K_b is least of $\dfrac{45}{3\times(24+2)}$, $\dfrac{70}{3\times(24+2)} - 0.25$, $\dfrac{400}{410}$, 1.0

$\therefore \quad K_b = 0.577$

∴ Bearing strength on 7.6 mm web of column

$$T_{db} = 2.5\times0.577\times24\times\frac{7.6\times410}{1.25}$$

$$= 86300 \text{ N}$$

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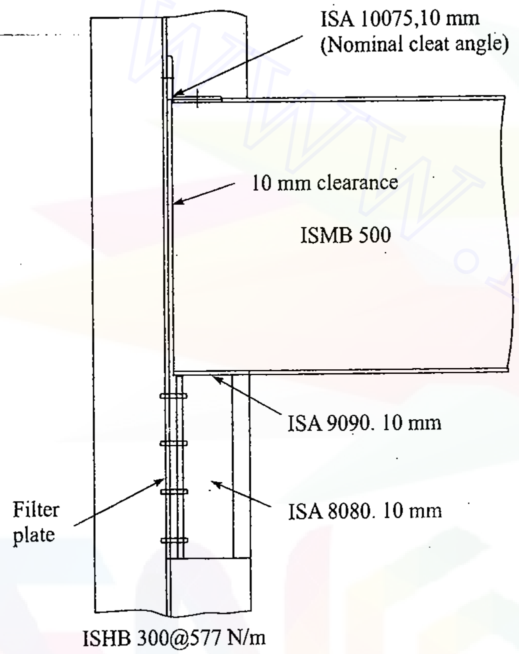

(a) Sectional Elevation

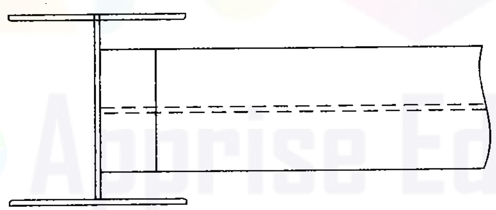

Figure 8.12

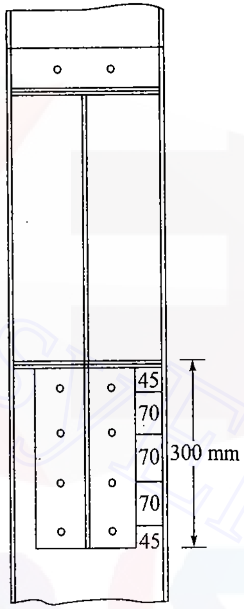

Figure 8.13

Bolt value in shear $V_{db} = 65192$ N

Tensile strength of M24 bolts:

$$= \frac{0.90 f_{ub} A_n}{\gamma_{mb}} < f_{yb} \frac{A_{sb}}{\gamma_{mo}}$$

$$A_n = 0.78 \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 24^2 = 352.864 \text{ mm}^2$$

$$\therefore \text{ Tensile strength } = \frac{0.90 \times 400 \times 352.864}{1.25} < 240 \times \frac{\pi}{4} \times 24^2 \times \frac{1}{1.1}$$

$$= 101625 < 98703 \text{ N}$$

Hence tensile strength $T_{db} = 98703$ N

Number of bolts:

Providing bolts in two rows at 70 mm pitch number of bolts per row $= \sqrt{\dfrac{6M}{2(V_{sb})p}}$

$$= \sqrt{\frac{6 \times 25.316 \times 10^6}{2 \times 65192 \times 70}}$$

$$= 4.08$$

Provide 4 bolts in each row as shown in the Fig. 8.13.

$$\therefore \text{ Direct shear force on each bolt} = \frac{375 \times 10^3}{8}$$

$$V_{sb} = 46875 \text{ N}$$

Assuming centre of gravity of flexural forces at $\frac{h}{7}$ from bottom side,

$$\frac{h}{7} = \frac{300}{7} = 42.86 \text{ mm}$$

i.e., neutral axis lies below first bolt which is at distance 45 mm from the lower edge of stiffener angle.

Bolt No.	1	2	3	4
y	2.14	72.14	142.14	212.14

Since there are two rows of bolts

$$\therefore \quad \Sigma y_i = 2 \times 428.56$$

$$\Sigma y_i^2 = 2 \times 70415.92$$

$$\therefore \quad M' = \frac{M}{1 + \dfrac{2h}{21}\dfrac{\Sigma y_i}{\Sigma y_i^2}} = \frac{24.098 \times 10^6}{1 + \dfrac{2 \times 300}{21} \times \dfrac{2 \times 428.56}{2 \times 70415.92}}$$

$$= 20.528 \times 10^6$$

$\therefore$ Tensile force in extreme bolt

$$T_b = \frac{M'y}{\Sigma y_i^2} = \frac{20.524 \times 10^6 \times 212.14}{2 \times 70415.92}$$

$$= 30922 \text{ N}$$

Check by interaction formula

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 = \left(\frac{46875}{65192}\right)^2 + \left(\frac{30922}{98703}\right)^2$$

$$= 0.615 < 1.0$$

Hence safe.

Use stiffener of length 370 mm with 4 bolts of 24 mm size on each side.

8.5 DESIGN OF SMALL MOMENT RESISTANT CONNECTIONS

If the moment to be transferred is small, clip angles may be provided to transfer moment and web angles to transfer shear [Ref. Fig. 8.14]. Hence the design consists of:

(a) Design of clip angle connections
(b) Design of web angle connections

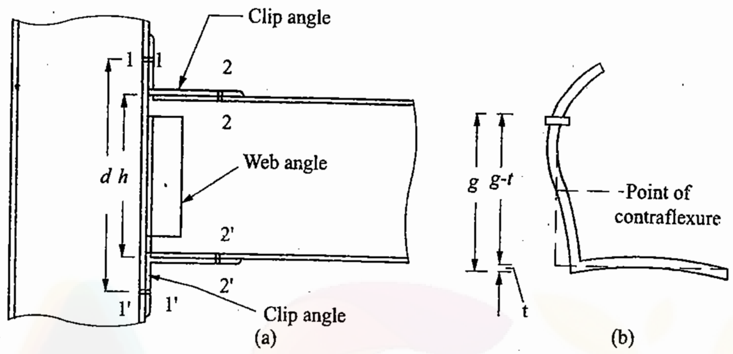

Figure 8.14

(a) Design of clip angle connection

(i) Force at (1) – (1), $P = \dfrac{M}{d}$. Bolts should be capable of resisting this force in tension

(ii) Moment in the leg of angle.

Referring to Fig. 8.14(b),

If, point of contraflexure is assumed at 0.5 $(g - t)$.

$$M = 0.5\, P\, (g - t).$$

More conservatively point of contraflexure may be assumed at 0.6 $(g - t)$ from bolt.

Then $$M = 0.6\, P\, (g - t).$$

Taking length of clip angle l = width of flange of beam connected, design moment of angle

$$M_d = \frac{l Z_p}{1.1} f_y$$

$$= l \frac{1}{4} t^2 f_y \times \frac{1}{1.1}$$

This should be more than M.

If thickness required is more than available the thickness is to be built up with addition of plates or split I-sections may be used as shown in Fig. 8.15.

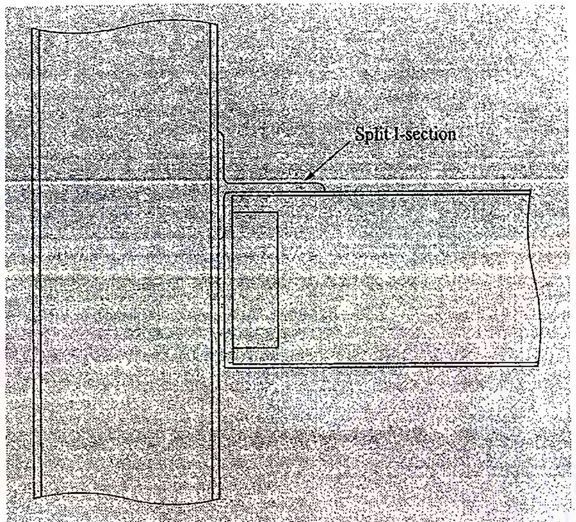

Figure 8.15

(iii) Design of connection between clip angle and beam (2-2):

The bolts are subjected to shear force of

$$V = \frac{M}{h}$$

Connection is to be designed to resist this shear force.

(b) Design of web angle connection: It is same as design of framed connection (Fig. 8.2)

The design procedure is illustrated with the example below.

Example 8.4

A beam ISMB 300 transmits an end shear of 120 kN and a moment of 20 kN-m to the flange of a column ISHB 300 @ 577 N/m. Using 20 mm dia shop bolts design suitable end connection.

Solution:

Factored end moment $M = 1.5 \times 20 = 30$ kN-m

Factored end shear $V = 1.5 \times 120 = 180$ kN

For ISMB 300,

$t_f = 12.4$ mm $\quad t_w = 7.5$ mm

For ISHB 300 @ 577 N/m,

$t_f = 10.6$ mm $\quad t_w = 7.6$ mm

Strength of bolts in single shear

$$= 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 45272 \text{ N}$$

Strength of bolts in double shear

$$= 1.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 103314 \text{ N}$$

Strength of bolt in tension

$$= 0.78 \times \frac{\pi}{4} \times 20^2 \times 0.9 \times \frac{400}{1.25} < \frac{\pi}{4} \times 20^2 \times \frac{240}{1.1}$$

$$= 70572 < 68544 \text{ N}$$

Strength of bolt in tension = 68544 = 68.544 kN

Strength in bearing:

Assuming edge distance $e = 40$ mm, $p = 60$ mm,

K_b is the least of $\frac{e}{3(20+2)}, \frac{p}{3(20+2)} - 0.25, \frac{400}{410}, 1.0$

i.e., $K_b = 0.606$

∴ Bearing strength on web of ISMB 300

$$= 2.5 \times 0.606 \times 20 \times 7.5 \times \frac{410}{1.25}$$

$$= 74538 \text{ N}$$

Bearing strength on flange of ISHB 300, @ 577 N/m $= 2.5 \times 0.606 \times 20 \times 10.6 \times \frac{410}{1.25}$

$$= 105347 \text{ N}$$

(i) Force P on bolt connecting clip angle to column:

Provide ISA 200 mm × 100 mm angle as clip angle. Then distance between gauge lines of bolts

$$d = 300 + 2 \times 60 = 420 \text{ mm}$$

∴ Pull on two bolts

$$P = \frac{30 \times 10^6}{420} = 71.429 \text{ kN.}$$

Allowable pull on two bolts = $2 \times 68.544 > P$

Hence adequate.

(ii) Moment in the angle, assuming $t = 15$ and $g = 60$ mm is $M = 0.6P\left(g - \frac{t}{2}\right) = 0.6 \times 71.429$ $(60 - 7.5) = 2250$ kN-mm.

Width of flange of beam ISMB 300

$l = 140$ mm

$\therefore$ Thickness of clip angle 't' is such that

$$\frac{1}{4} l t^2 f_y \times \frac{1}{\gamma_{mo}} = M$$

$$\frac{1}{4} \times 140 \times t^2 \times 250 \times \frac{1}{1.1} = 2250 \times 10^3$$

$\therefore$ $t = 16.818$ mm.

But available maximum thickness is only 15 mm.

$\therefore$ Either provide ISA 200100, 12 mm with 6 mm plate or go for ISA200150, 18 mm angle or use split section of ISMB 450 which has flange thickness 17.4 mm.

(iii) Horizontal shear in bolts connecting clip angle to beam

$$P = \frac{\text{Moment in connection}}{\text{Depth of beam}}$$

$$= \frac{30 \times 10^6}{300} = 100 \times 10^3 \text{ N} = 100 \text{ kN}$$

Strength of bolt in single shear = 45.272 kN

$\therefore$ Number of bolts required $= \dfrac{100}{45.272} = 2.2$

Provide 4 bolts in two rows

Design of web angle

Bolt value of bolts connecting web of beam and the angle is smaller of 103314 N and 74538 N

$\therefore$ $R = 74538$ N

$\therefore$ The number of bolts required $= \dfrac{180 \times 10^3}{74538} = 2.41$

Provide 3 M20 bolts with minimum edge distance of 45 mm and pitch of 60 mm.

Bolts connecting angle to web of column are in single shear. Their strength is smaller of 45272 N and 105347 N. i.e.,

$$R = 45272 \text{ N} = 45.271 \text{ kN}$$

Number of bolts required $= \dfrac{180}{45.272} = 3.98$

Provide 6 bolts in two rows (one on either side of web of beam). Minimum edge distance 45 mm and at a pitch 60 mm, Use ISA 10075, 8 mm.

Figure 8.16 shows the details of connection.

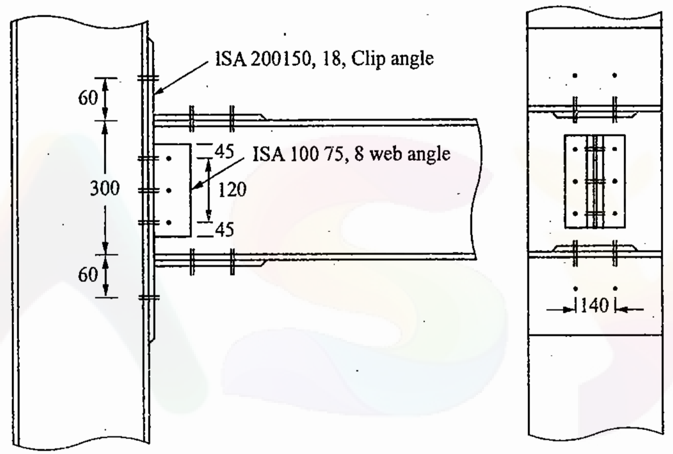

Figure 8.16

8.6 DESIGN OF LARGE MOMENT CONNECTIONS

If end moment to be transferred from beam to column is large, clip angles are not sufficient. In such cases bracket type moment resistant connections are used. Figure 8.17 shows a typical bracket type connection.

In this type a bracket plate, which has thickness equal to that of web of the beam is connected to flange of the beam and to the flange of column using horizontal and vertical angles. If exact thickness of plate is not readily available slightly larger plate is taken and machined to the size of web. Design of such connection consists in designing

(a) Vertical angle to resist moment $M = 0.6P(g - t)$
(b) Bolts A-A to resist direct shear and tension
(c) Bolts B-B to resist vertical shear and horizontal force due to moment
(d) The bracket plate
(e) Horizontal angle
(f) Bolts C-C to resist shear and tension
(g) Bolts D-D to resist vertical shear and horizontal force due to bending.

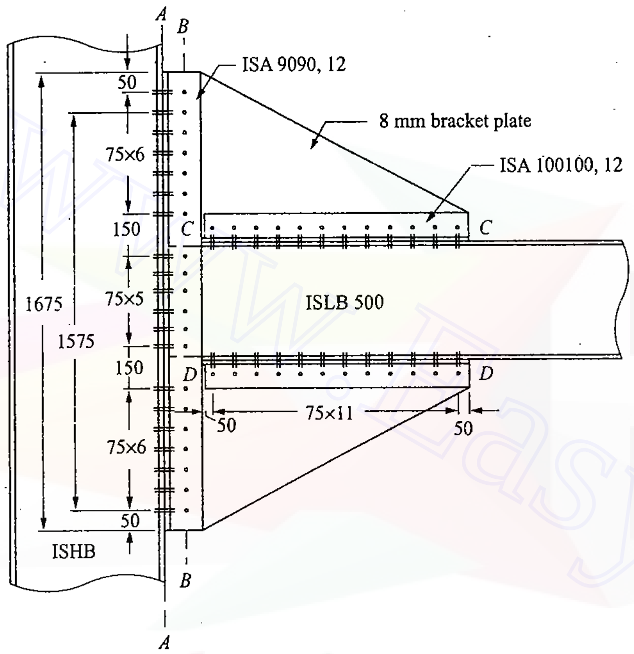

Figure 8.17

The design procedure is illustrated with the example below:

Example 8.5

Design a bracket connection to connect a beam ISLB 500 to a column ISHB 400 @ 806 N/m, if vertical shear and moment to be transmitted are 120 kN and 130 kN-m respectively. Use M24 bolts at a pitch of 75 mm. Provide edge distance of 50 mm for all connections.

Solution:

End shear = 120 kN

End moment = 130 kN-m

$\therefore$ Factored end shear = 1.5 × 120 = 180 kN

Factored end moment = 1.5 × 130 = 195 kN-m

For beam, $t_w = 9.2$ mm $\quad t_f = 14.1$ mm

For column, $t_f = 12.7$ mm

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Strength of M24 bolts,

in single shear $= 0.78 \times \frac{\pi}{4} \times 24^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 65192$ N

in double shear $= 1.78 \times \frac{\pi}{4} \times 24^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 148772$ N

in tension $= 0.78 \times \frac{\pi}{4} \times 24^2 \times 0.9 \times \frac{400}{1.25} < \frac{\pi}{4} \times 24^2 \times \frac{240}{1.1}$, since $f_{yg} = 240$ N/mm^2

$= 98703$ N

Strength in bearing with $e = 50$ mm and $p = 75$ mm

K_b is the least of $\frac{50}{3(24+2)}$, $\frac{75}{3(24+2)} - 0.25$, $\frac{400}{410}$, 1.0

$\therefore \quad K_b = 0.641$

$\therefore$ Bearing strength on 9.2 mm plate (plate slightly more than that of web of beam)

$$= 2.5 \times 0.641 \times 24 \times 9.2 \times \frac{410}{1.25} = 116057 \text{ N}$$

Bolt value:

For bolts A-A which are in single shear and tension is $R_1 = 65192$ N

For bolts B-B which are in double shear and bearing on bracket plate

$$R_2 = 116057 \text{ N}$$

For bolts C-C which are in single shear and tension, but field connected,

$$R_3 = 65192$$

For bolts D-D which are in double shear and bearing over bracket plate

$$R_4 = 116057 \text{ N}$$

(a) *Design of vertical angle*:

Using 9090 angle and taking $g - t = 45$ mm

$$M = 0.6\,P\,(g - t)$$

Maximum P that can be allowed is bolt value of 65192 N.

Then $M = 0.6 \times 65192 \times 45 = 1.76 \times 10^6$ N-mm

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Strength of angle per pitch width of 75 mm is $= \frac{1}{4} \times 7.5 \times t^2 \frac{f_y}{1.1} = \frac{1}{4} \times 75 \times t^2 \times \frac{250}{1.1}$

Equating this moment of resistance to moment

we get
$$\frac{1}{4} \times 75 \times t^2 \times \frac{250}{1.1} = 1.76 \times 10^6$$
$$t = 20.32 \text{ mm}$$

Maximum thickness available is only 12 mm. Hence we can reduce P by restricting bolt value R.

$$0.6 \times R \times 45 = \frac{1}{4} \times 75 \times 12^2 \times \frac{250}{1.1}$$
$$R = 22727 \text{ N}$$

(b) *Design of bolts A-A*:

$R = 22727$ N, $M = 195$ kN-m

Since there are two rows (one on either side of bracket plate) number of bolts required is given by

$$n = \sqrt{\frac{6M}{2pR}} = \sqrt{\frac{6 \times 195 \times 10^6}{2 \times 75 \times 22727}}$$
$$= 18.53$$

Provide 20 bolts in each row as shown in Fig. 8.17.

$h = 1575 + 50 = 1625$ mm. $\therefore$ $h/7 = 232.14$ mm.

(i) *Check for Bolts A-A*:

N-A is between 3rd and 4th bolt. Distance of 4th bolt from N-$A = 50 + 75 \times 3 - 232.14 = 42.86$ mm

Bolt No.	y in mm	Bolt No.	y in mm
4	42.86	13	717.86
5	117.86	14	792.86
6	192.86	15	867.86
7	267.86	16	942.86
8	342.86	17	1017.86
9	417.86	18	1092.86
10	492.86	19	1167.86
11	567.86	20	1242.86
12	642.86		

Since these are two rows of bolts

$$\therefore \quad \Sigma y_i = 2 \times 7426.04 \text{ mm}$$
$$\Sigma yi^2 = 2 \times 9320572 \text{ mm}^2$$

$$M' = \frac{M}{1 + \frac{2h}{21} \frac{\Sigma y_i}{\Sigma y_i^2}} = \frac{195 \times 10^6}{1 + \frac{2 \times 1625}{21} \times \frac{2 \times 7426.04}{2 \times 9320572}}$$
$$= 173.59 \times 106 \text{ N-mm}$$

Tension in extreme bolts

$$= \frac{173.59 \times 10^6 \times 1242.86}{2 \times 9320572}$$
$$= 11574 \text{ N}$$

Shear force on each bolt $= \frac{180 \times 10^3}{2 \times 20} = 4500$ N

$\therefore$ Interaction formula is

$$\left(\frac{4500}{65192}\right)^2 + \left(\frac{11574}{22727}\right)^2 = 0.264 < 1$$

Hence adequate.

(c) *Check for bolts B-B*:

For these bolts, bolt value $R = 116057$ N

These bolts are in a single line and are subject to moment in their plane.

$\therefore$ Number of bolts required

$$n = \sqrt{\frac{6M}{pR}} = \sqrt{\frac{6 \times 195 \times 10^6}{75 \times 116057}} = 11.59$$

Here also provide 20 bolts [Ref. Fig. 8.17]. In this case $k_i = 0$: $r_i = y_i$ N-A is at mid depth. Hence $r_i = y_i = 37.5$, 112.5, 187.5, 337.5, 412.5, 487.5, 562.5, 637.5, 712.5, 787.5,

$$\Sigma y_i^2 = 2 \times 2421562.5$$

$\therefore$ Force in extreme bolt due to moment

$$= \frac{195 \times 10^6 \times 787.5}{2 \times 2421562.5} = 31707 \text{ N, Horizonal}$$

Direct shear force $= \dfrac{180 \times 10^3}{20} = 9000$ N, vertical

$\therefore$ Resultant force $= \sqrt{31707^2 + 9000^2}$

$= 32960 \text{ N} < 116097 \text{ N}$

Hence adequate.

(d) *Design of Bracket plate*:

Length of bracket plate = 1675 mm Let thickness be 8 mm

No. of holes for M24 bolts = 20

Diameter of each bolt hole = 24 + 2 = 26 mm

$\therefore$ Effective length along bolt line B-B

$= 1675 - 20 \times 26 = 1155$ mm.

Bending resistance along this section

$$= \frac{1}{4} \times 8.0 \times 1155^2 \times \frac{250}{1.1} = 606.375 \times 10^6 \text{ N-mm}$$

$$= 606.375 \text{ kN-m} > 195 \text{ kN-m}$$

Hence adequate.

(e) *Horizontal Angle*:

Here also $P = R = 22727$ N may be maintained, so as to use 12 mm thick angle. Provide 2ISA 100100, 12 mm.

(f) *Design of Bolts Along C-C*:

Force on extreme bolt along B-B = 31707 N

Force on nearest bolt (bolt No. 4) from neutral axis in line BB on top bracket, which is at a distance of 337.5 mm from neutral axis is

$$= 31707 \times \frac{337.5}{787.5} = 13589 \text{ N.}$$

$\therefore$ Resultant of horizontal forces of all bolts above top bracket $= \dfrac{31707 + 13589}{2} \times 7 = 158536$ N

Line of action of this force from top bolt

$$= \frac{450}{3} \frac{31707 + 2 \times 13589}{31707 + 13589} = 195 \text{ mm}$$

Its distance from line C-C

$= 537.5 - 195 = 342.5$

Hence moment at the level of bolts along C-C

$= 158536 \times 342.5 = 54.30 \times 10^6$ N-mm

Here also allowable maximum tension

$P = 22717$ N

$\therefore$ Number of bolts required in each row

$$n = \sqrt{\frac{6 \times 54.3 \times 10^6}{2 \times 75 \times 22727}} = 9.478$$

Provide 12 bolts to connect horizontal angle to the beam.

Check for combined action of shear and Tension:

Length of angle = 50 + 11 × 75 + 50 = 925 mm

Distance of last bolt from the beginning of angle = 925 − 50 = 875 mm

Assuming neutral axis is at $\frac{1}{7}$th of this length i.e., at $\frac{875}{7} = 125$ mm, we find it coincides with the position of 2nd bolt. Hence

Bolt No.	3	4	5	6	7	8	9	10	11	12
(y)	75	150	225	300	375	450	525	600	675	750

$\therefore$ $\Sigma y_i^2 = 2 \times 2165625 \text{ mm}^2$

Since there are two bolts at every 'y' distance

Total $\Sigma y^2 = 2 \times 2165625 \text{ mm}^2$

$\therefore$ Maximum tensile force in extreme bolt

$$= \frac{M}{\Sigma y_i^2} y_{12} = \frac{54.30 \times 10^6}{2 \times 2165625} \times 750$$

$$= 9402 \text{ N}$$

Direct shear force $= \dfrac{158536}{2 \times 12} = 6606$ N

$\therefore$ Interaction formula gives,

$$\left(\frac{6606}{65192}\right)^2 + \left(\frac{9402}{22727}\right)^2 = 0.181 < 1.0$$

Hence adequate.

(g) *Design of Bolts Along D-D:*

Adopting a gauge of 50 mm from the end of angle, the distance of the resultant horizontal force from the line D-D

$$= 342.5 - 50 = 292.5$$

The horizontal force is = 158536 N [As found in the design of bolts along C-C]

$\therefore$ Moment to be resisted by these bolts

$$M = 158536 \times 292.5 = 46.372 \times 10^6 \text{ N-mm}$$

$\therefore$ Number of bolts required along D-D

$$n = \sqrt{\frac{6 \times 46.372 \times 10^6}{1 \times 75 \times 116057}}, \text{ since } R = 116057 \text{ N for these bolts}$$

$$= 5.65$$

Provide 12 bolts in this row also.

Force due to bending $= \dfrac{M}{\Sigma y_i^2} y_{12}$

$\Sigma y_i^2 = 2165625$, since there is only one bolt at every y specified.

$$\therefore \quad F_2 = \frac{46.372 \times 10^6}{2165625} \times 750 = 16059 \text{ N}$$

Direct shear force $F_1 = \dfrac{180 \times 10^3}{12} = 15000$ N.

$\therefore$ Interaction formula is

$$\left(\frac{15000}{65192}\right)^2 + \left(\frac{16059}{22727}\right)^2 = 0.552 < 1.0$$

Hence adequate.

Connection details are shown in Fig. 8.17

* * *

Questions

1. Draw the typical sketches to show the following beam column connections:
   (a) Framed connection
   (b) Unstiffened seated connection
   (c) Stiffened seated connection
   (d) Clip-Angle connection
   (c) Bracket type moment resistance connection. Also state when do you prefer those end connections.
2. An ISMB 300 transmits an end reaction of 180 kN to the web of an ISMB 450. Design a framed connection. Show the details with a neat sketch.
3. Design a seat connection for a beam end reaction of 90 kN. The beam is ISMB 250 connected to the flange of the column section ISHB 200 @ 392 N/m.
4. An ISMB is connected to the flange of an ISHB 250 @ 537 N/m. Design a stiffened seated connection.
5. A beam ISMB 250 transmit an end shear of 80 kN and a moment of 16 kN-m to an ISHB 250 @ 537 N/m. Using 20 mm bolts design a clip angle connection.
6. Design a connection between beam ISMB 450 and the flange of column ISHB 300 @ 577 N/m to transmit a shear force of 100 kN and a moment of 120 kN-m. Use M24 bolts at 75 mm pitch.

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9

DESIGN OF WELDED BEAM CONNECTIONS

Beams may be connected to the supporting beam or to the supporting column by welding. In fact welded connections are used more commonly instead of bolted connections. The end of the beam may be designed to transfer only shear to the supporting structure by

(a) Framed connection
(b) Unstiffened seated connection or
(c) Stiffened seated connection.

The ends of beam may be designed to transfer shear as well as moment by welded connection. Such connections are known as moment resistant connection.

In this chapter, the design of all such connections are explained and illustrated with examples.

9.1 FRAMED CONNECTIONS

The direct fillet or butt welds explained in chapter 4 need accurate lengths and edge finishes, which may be quite difficult to achieve. Instead of these connections, framed connection may be adopted which are flexible. The following two types of framed connections are possible.

(i) Double Plated Framed Connections
(ii) Double Angle Framed Connections.

9.1.1 Double Plated Framed Connection

Figure 9.1 shows a typical double plated framed connection. In this type of connection 50 mm wide two plates of depth $\frac{1}{2}$ to $\frac{2}{3}$ of beam, with thickness 1.5 mm more than that of web of the beam are used. To allow rotation at the end of the beam it is necessary to limit the depth of plate to $\frac{2}{3}$ of beam. One plate is shop welded to the beam and another plate is shop welded to column/supporting beam. Beam is kept on erection seat and then field welding is done to complete the connection. Sometimes instead of providing erection seat, erection bolts are provided to temporarily support beam for field welding.

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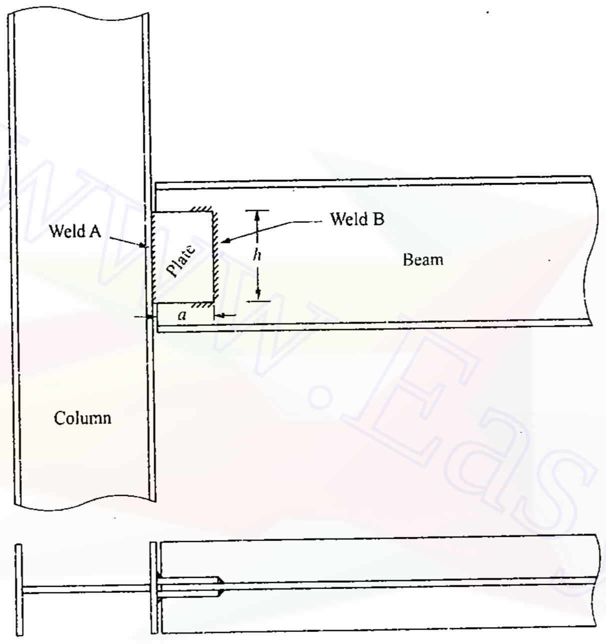

Figure 9.1

Weld B is kept about $\frac{4}{5}$ th of web of beam in size. The welds are returned by about 12 mm. The depth 'h' of weld B is kept such that it can resist shear V and moment $V \times a$ safely, where a is width of connecting plate (which is usually 50 mm).

Weld A of depth d is designed to resist shear V only.

The design procedure of such connection is illustrated with the example below:

Example 9.1

An ISMB 400 beam is to be connected to an ISHB 250 @ 537 N/m to transfer end force of 140 kN. Design double plated welded connection.

Solution:

Factored $V = 140 \times 1.5 = 210$ kN

Using 50 mm wide plates, factored moment on weld connecting plate and web of beam (weld B)

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$$M = 210 \times 50 \text{ kN-mm} = 210 \times 50 \times 10^3 \text{ N-mm.}$$

Thickness of plate should be 1.5 mm more than the web thickness of the beam.

$\therefore$ Thickness of plate $= t_w + 1.5 = 8.9 + 1.5 = 10.4$ mm.

Use 12 mm plates.

Since one weld is shop weld and the other is field weld, design is made for field weld and the same is adopted for shop weld also. For field weld partial safety factor $\gamma_{mw} = 1.5$. Hence

$$\text{Strength of weld} = \frac{f_u}{\sqrt{3}} \times \frac{1}{1.5} = \frac{410}{\sqrt{3}} \times \frac{1}{1.5}$$

$$f_{wd} = 157.81 \text{ N/mm}^2$$

Design of Weld B:

$V = 210$ kN; $M = 210 \times 50 \times 10^3$ N-mm. Assuming 6 mm as the size of weld, throat thickness of weld $= 0.7 \times 6$. Since there are two rows of welds,

$$d = \sqrt{\frac{6M}{2 \times t \times f_{wd}}} = \sqrt{\frac{6 \times 210 \times 50 \times 10^3}{2 \times 0.7 \times 6 \times 157.81}}$$

$$= 218 \text{ mm.}$$

The above depth is required to resist bending alone. Since the weld has to resist shear also, try 15 to 20% additional depth.

Trial depth $h = 1.2 \times 218 = 261.6$ mm

Try $h = 260$ mm

Selected h is between $\frac{1}{2}$ to $\frac{2}{3}$ rd of depth of beam.

$$\therefore \quad \text{Shear due to bending } q_2 = \frac{M}{Z} = \frac{6M}{2th^2}$$

$$= \frac{6 \times 210 \times 50 \times 10^3}{2 \times 0.7 \times 6 \times 260^2} = 110.95 \text{ N/mm}^2$$

$$\text{Direct shear stress } q_1 = \frac{V}{2th} = \frac{210 \times 10^3}{2 \times 0.7 \times 6 \times 260}$$

$$= 96.15 \text{ N/mm}^2$$

$$\therefore \quad \text{Resultant stress } q = \sqrt{110.95^2 + 96.15^2}$$

$$= 146.8 \text{ N/mm}^2 < 157.81 \text{ N/mm}^2$$

Hence adequate. Provide 6 mm size fillet welds, 260 mm long.

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Design of Weld A:

This weld carries only shear.

$$V = 210 \text{ kN}$$

The length of this weld is also kept 260 mm.

Let size of the weld be s.

$\therefore$ Throat thickness of weld $t = 0.7\,s$. Since there are two weld lines equating strength of these welds to shear, we get

$$2\,t\,hf_{wd} = V$$

$$2 \times 0.7\,s \times 260 \times 157.81 = 210 \times 10^3$$

$\therefore$ $s = 3.65$

Provide 4 mm welds.

9.1.2 Double Angle Framed Connection

Instead of two plates, two angles may be used for connecting beam and the supporting member to get more flexibility in the connection. The angles are connected to the web of the beam by shop welding. The length of the leg of angle used for this is usually 60 mm. These welds are provided on all the three sides of the leg of angle as shown in the Fig. 9.2. Depth of the angle used is kept $\frac{1}{2}$ to $\frac{2}{3}$ the depth of beam to achieve required flexibility in the connection. In the figure these welds are referred as weld B.

The leg of the angle to be connected to supporting member is kept 80 to 90 mm. They are connected to the supporting structure by field welding. To facilitate field welding erection bolts are provided in the position as low as possible. The thickness of connecting angles should be $\frac{4}{3}\times$ the size of weld used in beam, so that the required size of the weld is provided along the rounded edge of the angle.

Design of Shop Welds B:

Assuming 10 mm gap between the edge of the beam and the face of the column, the shape of the weld is as shown in Fig. 9.3. Since there is one such weld on each face of web of the beam, reaction resisted by each weld is $\frac{R}{2}$. As end on the face of the column is assumed simple support, the weld is subjected to a torsional moment $\frac{R}{2}\times e_2$ also, where e_2 is the eccentricity of the c.g. of the weld. These shop welds are designed to resist a vertical shear of $\frac{R}{2}$ and moment $\frac{R}{2}\times e_2$.

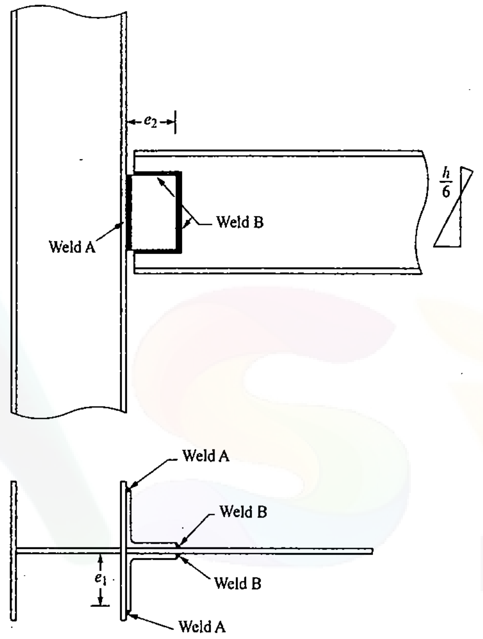

Figure 9.2

Design of Field Welds A:

The welds transfer vertical shear force of $\frac{R}{2}$ at an eccentricity of e_1 from the face of web of the beam. In the compression zone entire contact area of angle resist the force while in tensile zone only weld has to resist the force. Hence neutral axis is very close to compression edge. This distance may be assumed as $\frac{h}{6}$, if h is the depth of weld. Referring to Fig. 9.4, the horizontal shear in the weld A may be calculated as,

$$\left(\frac{1}{2} q_{sh} \times t \times \frac{5}{6} h\right) \times \frac{2}{3} h = \frac{R}{2} e_1$$

$$\therefore \quad q_{sh} = \frac{9}{5} \frac{R e_1}{t h^2}$$

The vertical shear is

$$q_v = \frac{R/2}{t\,h} = \frac{R}{2 \times 0.7\,s\,h}$$

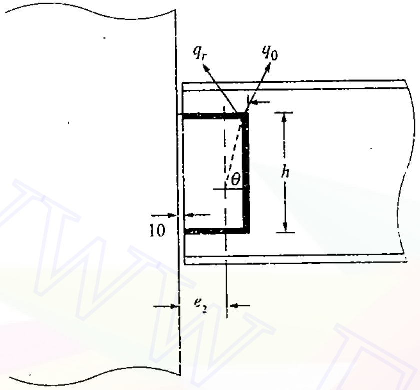

Figure 9.3

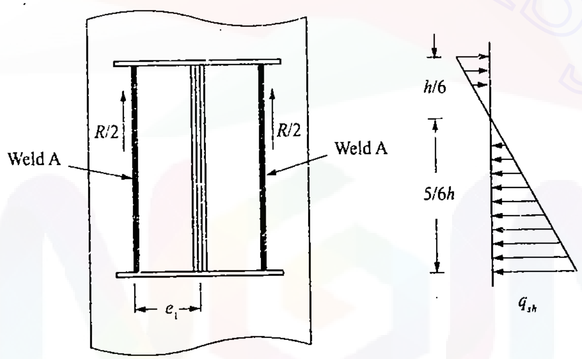

Figure 9.4

The resultant shear stress $q = \sqrt{q_{sh}^2 + q_v^2} \leq$ the permissible value in field weld.

From this condition thickness of weld and hence size of the weld is found. To provide the required size of weld at the edge of angle, the angle should have thickness $\frac{4}{3}s$.

The design procedure is illustrated with the example below:

Example 9.2

Design a double angle framed connection for the problem given in example 9.1.

Solution:

Depth of beam = 400 mm.

Use 2 ISA 9060 as frame angles.

Let the depth of angle be 260 mm.

(a) *Design of Field Welds A:*

Factored reaction $R = 1.5 \times 140 = 210$ kN

Factored reaction on each weld = 105 kN

$e_1 = 90$ mm.

$\therefore$ Horizontal shear

$$q_{sh} = \frac{9}{5}\frac{Re_1}{th^2} = \frac{9}{5} \times \frac{210 \times 90 \times 10^3}{260^2 t}$$

$$= \frac{503.25}{t} \text{ N/mm}^2$$

Vertical shear

$$q_v = \frac{210 \times 10^3}{2t \times 260} = \frac{403.85}{t}$$

Strength of field weld

$$= \frac{f_u}{\sqrt{3}} \times \frac{1}{1.5} = \frac{410}{\sqrt{3} \times 1.5} = 157.81 \text{ N/mm}^2$$

Equating resultant stress to it, we get

$$= \sqrt{\left(\frac{503.25}{t}\right)^2 + \left(\frac{403.85}{t}\right)^2} = 157.81$$

$$\therefore \quad t = \frac{645.26}{157.81} = 4.08 \text{ mm}$$

$$\therefore s = \frac{4.08}{00.7} = 5.82$$

Provide 6 mm weld.

Design of Shop Welds B:

Strength of shop weld $= \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$

$= 189.37$ N/mm^2

Let 't' be the throat thickness of the weld.

Distance of c.g. of the weld from the vertical weld $= \dfrac{2\times 50t\times 50/2}{2\times 50t+260t} = 6.94$ mm

$$\therefore \quad e_2 = 60 - 6.94 = 53.06 \text{ mm}$$

Weld on each side has to resist $V = \dfrac{210}{2} = 105$ kN

and $M = 105\times 53.06 = 5571.3$ kN-mm $= 5571.3\times 10^3$ N-mm

Properties of Weld B:

$$I_{zz} = \frac{1}{12}\times t\times 260^3 + 2 + 50\,t\times 130^2 = 3154667\,t \text{ mm}^4$$

$$I_{yy} = 260\,t\times 6.94^2 + 2\left[\frac{1}{12}t\times 50^3 + 50t(25-6.94)^2\right]$$

$$= 65972\,t \text{ mm}^4$$

$$I_p = I_{zz} + I_{yy} = 3154667t + 65972\,t = 3220639\,t \text{ mm}^4$$

$\therefore$ Stress due to twisting moment

$$q_r = \frac{M}{I_p}\times r$$

$$\text{Now } r = \sqrt{6.94^2+130^2} = 130.185$$

$$\therefore \quad q_r = \frac{5571.3\times 10^3}{3220639t}\times 130.185 = \frac{225.2}{t}$$

Direct shear

$$q_v = \frac{105\times 10^3}{(2\times 50+260)t} = \frac{291.67}{t}$$

$$\theta = \tan^{-1}\frac{130}{6.94} = 86.944°$$

$\therefore$ Resultant shear $= \sqrt{q_v^2 + q_r^2 + 2q_v q_r\cos 86.944°}$

$$= \frac{377.876}{t} \text{ N/mm}^2$$

Equating it to weld strength, we get

$$\frac{377.876}{t} = 189.37$$

$$\therefore t = 2.0 \text{ mm}$$

$$\therefore s = \frac{t}{0.7} = \frac{2}{0.7}\ 2.85 \text{ mm}$$

Use 3 mm weld.

Weld A required is 6 mm thick. Hence minimum thickness of angle required is $\frac{4}{3}\times 6 = 8$ mm Therefore use ISA 9060, 8 mm.

9.2 WELDED UNSTIFFENED SEAT CONNECTION

When the end reaction to be transferred is low, welded unstiffened seat connection may be used. The beam is placed over a seat angle and welded. To prevent lateral displacement a cleat angle ISA 100100, 6 mm may be placed and welded with 6 mm welds.

The bearing length of beam may be taken same as used for bolted connection (chapter 8). Thus the bearing length

$$b = \left(B - \sqrt{3}\,h_2\right) \nless \frac{B}{2}$$

where $B = \dfrac{F}{f_p t_w}$, $f_b = 187.5$ N/mm^2 for E250 steel.

A clearance of 10 mm is provided between the end of beam and column. Assuming reaction is distributed uniformly over bearing length 'b', the position of centroid of reaction is found. The thickness of angle is determined from the required bending strength of angle at the root of the angle.

Then the moment at the weld is found at the critical section of the weld. The resultant shear stress in the weld of size 't' due to vertical shear and horizontal shear is found and equated to strength of the weld in shear to get required throat thickness of the weld.

The procedure is illustrated below with the example:

Example 9.3

An ISMB 400 transfers an end reaction of 160 kN to the flange of an ISHB 300 @ 577 N/m. Design an unstiffened welded seat connection. Take $f_b = 0.75\times 250 = 187.5$ N/mm^2.

Solution:

For ISMB 400,

Width of the flange, $b_f = 140$ mm, $t_f = 16.0$ mm

$t_w = 8.9$ mm $\quad r_1 = 14$ mm.

$$B = \frac{F}{f_p t_w} = \frac{160 \times 10^3}{187.5 \times 10.2} = 84.8 \text{ mm}$$

$$b = B - \sqrt{3}(h_2) = 84.8 - \sqrt{3}\,(32.8)$$

$$= 32.8 \text{ mm} \quad \not< \frac{B}{2}$$

$$\therefore \quad b = \frac{84.8}{2} = 42.4 \text{ mm}$$

It is as shown in Fig. 9.5.

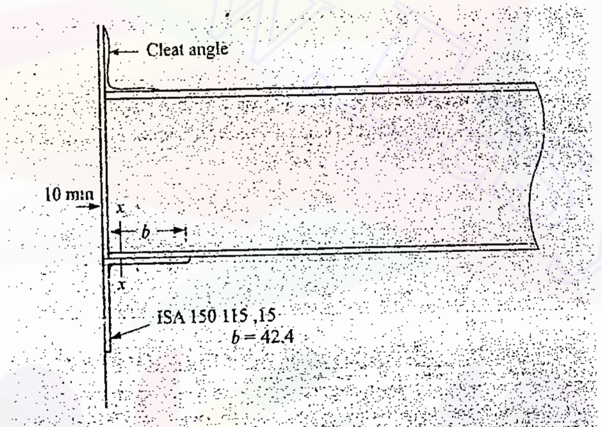

Figure 9.5

(a) *Design of Seating Angle:*

Assuming 15 mm thick angle of size 150115, the distance of end reaction from critical section

$$= 10 + \frac{1}{2} \times 42.4 - (15 + 11)$$

$$= 5.2 \text{ mm}$$

$\therefore$ Bending moment at the critical section in the angle = $160 \times 10^3 \times 5.7$ N-mm

Factored moment = $1.5 \times 160 \times 10^3 \times 5.2$ N-mm

Length of seating angle = 140 mm

$\therefore$ Moment of resistance

$$M_d = \frac{f_y Z_p}{\gamma_m} = \frac{250 \times \frac{1}{4} \times 140 \times t^2}{1.25} + 7000t^2$$

Equating it to the factored moment, we get

$$7000t^2 = 1.5 \times 160 \times 10^3 \times 5.2$$

$$\therefore \quad t = 13.35 \text{ mm}$$

Hence use 15 mm thick angle.

(b) *Design of weld:*

Length of vertical weld = 150 mm

Factored shear force = 1.5×160 kN

If 't' is the throat thickness of weld,

$$q_v = \frac{1.5 \times 160 \times 10^3}{2 \times 150 \times t} = \frac{800}{t}$$

The distance of end reaction from this weld

$$= \frac{42.4}{2} + 10 = 31.2 \text{ mm}$$

M at weld = $160 \times 10^3 \times 31.2 = 4992 \times 10^3$ N

Factored moment = $1.5 \times 160 \times 31.2 \times 10^3 = 7488 \times 10^3$ N-mm.

Hence horizontal shear

$$q_h = \frac{7488 \times 10^3}{2 \times \frac{1}{6} \times t \times 150^2} = \frac{998.4}{t}$$

$$\therefore \quad \text{Resultant shear stress} = \sqrt{\left(\frac{800}{t}\right)^2 + \left(\frac{998.4}{t}\right)^2}$$

$$= \frac{1279.4}{t}$$

Strength of weld $= \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = 189.37$ N/mm

Equating resultant shear stress to it, we get

$$\frac{1279.4}{t} = 189.37$$

$$t = 6.75 \text{ mm}$$

$$\therefore \quad \text{Size of weld } s = \frac{t}{0.7} = \frac{6.75}{0.7}$$

$$= 9.65 \text{ mm}$$

Provide 10 mm weld.

Since angle thickness is 15 mm, 10 mm weld can be provided at the edge of angle $\left(s \ngtr \frac{3}{4}15\right)$. Hence selected angle is ISA 150 × 115, 15 mm and is connected by 10 mm welds. At the top cleat angle ISA 100100, 6 mm may be field welded with 6 mm weld.

9.3 STIFFENED WELDED SEAT CONNECTIONS

Figure 9.6(a) shows a typical stiffened seat connection. The seat used may be a split I beam or two plates forming a T-section. The seat plate thickness is not less than the thickness of flange of beam and the thickness of stiffening plate is not less than the thickness of the web of beam. Seat plate and stiffening plate are welded as shown in Fig. 9.6(a). The width of seat plate is kept equal to the width of flange of beam. The same size plate may be used as stiffening plate. Weld looks like a T-section as shown in the Fig. 9.6(b).

The bearing length 'b' is calculated as explained for unstiffened connection, but is measured from outer end of seat plate. 10 mm clearance is provided between the end of the beam and the flange of the column. The size of weld is so selected that it can resist combined vertical and horizontal shear.

ISA 100100, 6 mm is used as clip angle at the top. It is welded with 4 mm size weld.

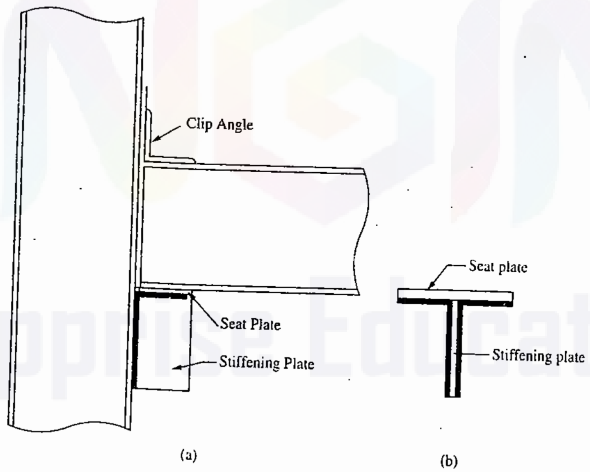

Figure 9.6

Example 9.4

Design a stiffened seat connection to connect the ISMB 500 transferring a load of 260 kN to an ISHB 300 @ 577 N/m.

Solution:

End reaction $F = 260$ kN

For ISMB 500,

$b_f = 180$ mm $\quad t_f = 17.2$ mm

$t_w = 10.2$ mm $\quad h_2 = t_f + r_1 = 17.2 + 17 = 34.2$ mm

$$\therefore \quad B = \frac{260 \times 10^3}{185 \times 10.2} = 137.8 \text{ mm}$$

$$\therefore \quad \text{Bearing length } b = B - \sqrt{3}\, h_2 \nless \frac{1}{2} B$$

$$= 137.8 - \sqrt{3} \times 37.95 \nless \frac{1}{2} \times 137.8$$

$$= 72.07 \text{ mm}$$

10 mm clearance is provided between the end of beam and the flange of column.

$\therefore$ Minimum width of plate required = 72.07 + 10 = 82.07 mm.

Use 90 mm seat plate. Its thickness should be at least equal to t_f = 17.2 mm. Hence use 18 mm thick plate. Width of this plate is kept equal to the flange width of beam = 180 mm.

Thickness of stiffening plate should be more than t_w (10.2 mm in this case).

Provide 12 mm plate. Let its depth also be 180 mm.

The distance of centre of gravity of reaction from column flange $= 90 - \frac{1}{2} \times 72.07 = 53.97$ mm.

$\therefore$ Factored bending moment $M = 1.5 \times 260 \times 53.97 = 21048$ kN-mm

$= 21.048 \times 10^6$ N-mm

Let throat thickness of weld be 't'. Figure 9.7 shows the shape of weld.

The distance of c.g. of weld from top

$$y_1 = \frac{2 \times 180 \times t \times 90}{168t + 2 \times 180t} = 61.4 \text{ mm}$$

$$\therefore \quad I_{xx} = 168t \times 61.4^2 + 2 \times \left[\frac{1}{12} \times t \times 180^3 + t \times 180(90 - 61.4)^2\right]$$

$$= 1899819t \text{ mm}^4$$

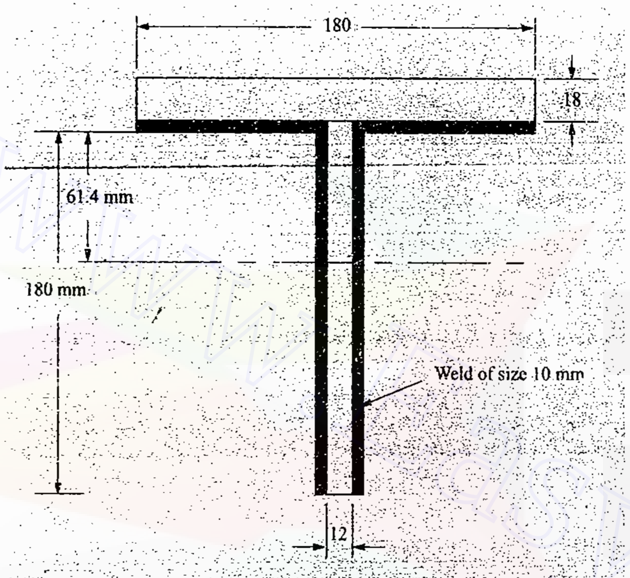

Figure 9.7

$\therefore$ Due to bending, horizontal shear in the weld is

$$q_h = \frac{M}{I}y = \frac{21.048\times10^6}{1899819t}\times61.4 = \frac{680}{t}\ \text{N/mm}^2$$

Direct shear stress

$$q_v = \frac{1.5\times260\times10^3}{2\times180t+168t} = \frac{738.6}{t}\ \text{N/mm}^2$$

$\therefore$ Resultant stress

$$q = \sqrt{q_h^2+q_v^2}$$

$$= \sqrt{\left(\frac{680}{t}\right)^2+\left(\frac{738.6}{t}\right)^2} = \frac{1003.96}{t}$$

Permissible stress in shop weld

$$f_w = \frac{410}{\sqrt{3}}\times\frac{1}{1.25} = 189.37\ \text{N/mm}^2$$

Equating the resultant stress to it, we get

$$\frac{1003.96}{t} = 189.37$$

$$\therefore\quad t = 5.30$$

$$\therefore\quad \text{Size of weld } s = \frac{t}{0.7} = \frac{5.30}{0.7} = 7.57\ \text{mm}$$

Provide 10 mm weld.

Provide clip angle ISA 100100, 6 mm at the top with 4 mm weld.

Size of seat plate = 90 × 180 × 18 mm

Size of stiffener plate = 180 × 90 × 12 mm

Size of shop weld = 10 mm.

9.4 MOMENT RESISTANT WELDED CONNECTIONS

If both moment and shear are to be transferred from beam to supporting structure, moment resistant connections are required. Such connections in welded structures are simpler compared to bolted structure. In this shear is transferred by any one of the following methods already discussed:

(a) Framed connection
(b) Unstiffened seated connection or
(c) Stiffened seated connection

To transfer moment, a tension plate connects top of flange of beam to the supporting structure as shown in Fig. 9.8.

Near the support, for the usual vertical downward loads, beam is in tension at the top and is in compression at bottom. The bottom flange of beam is connected to the supporting member by butt weld. The tension plate provided over top flange is connected to the supporting structure by full penetration butt weld. It needs a minimum gap of 4 to 5 mm between the plate and the column. A backing plate may be installed for welding efficiently. The width of tension plate is kept smaller than the flange width of beam so that welding on 3 sides is possible over the beam. The plate is kept sufficiently long so that a length of plate equal to its width is not welded. It helps in preventing failure of the joint before the failure of plate, which gives sufficient warning in the case of failure. A stiffener plate is welded in the supporting member to safeguard against web failure.

The following example gives the design procedure for the design of tension plate and its connection. For transferring shear a suitable method of connection should be designed as explained in the earlier part of this chapter.

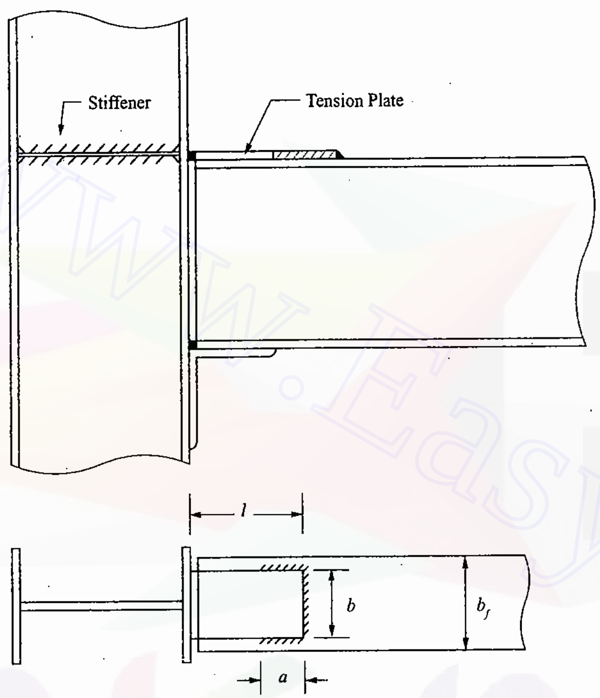

Figure 9.8

Example 9.5

An ISMB 400 transfers an end reaction of 160 kN and an end moment of 80 kN-m to the flange of an ISHB 300 @ 577 N/m. Design the moment resistant connection.

Solution:

Moment connection:

$$\text{Force in tension plate} = \frac{80\times10^3}{400} = 200 \text{ kN}$$

$$\therefore \quad \text{Factored force} = 200 \times 1.5 = 300 \text{ kN}$$

$$= 300 \times 10^3 \text{ N}$$

Design strength of the plate in tension

$$T_{dn} = 0.9\, A_n\, f_u \times \frac{1}{\gamma_{ml}}$$

$$= 0.9\, A_n \times 410 \times \frac{1}{1.25}$$

Equating it to the force, we get

$$0.9\, A_n\, 410 \times \frac{1}{1.25} = 300\times10^3$$

$$\therefore \quad A_n = 1016.2 \text{ mm}^2$$

Width of flange of ISMB 400 is 140 mm. Hence selecting width of tension plate = 110 mm, we get, thickness of tension plate

$$= \frac{1016.2}{110} = 9.24 \text{ mm}$$

Provide 110 mm wide and 10 mm thick flat. Connect it to the column by full penetration butt weld.

Full width of 110 mm and width 'a' on sides be welded with weld of thickness t. Then equating weld strength to tensile force, we get

$$(2a+110)t \times \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = 300\times10^3$$

$$\therefore \quad (2a+110)t = \frac{300\times10^3}{410} \times \sqrt{3} \times 1.25 = 1584.2$$

If s is the size of weld $t = 0.7s$

$$\therefore \quad (2a + 110)\, 0.7s = 1584.2$$

$$(2a+110)s = \frac{1584.2}{0.7} = 2263.1$$

Select weld size of 8 mm, we get

$$2a+110 = \frac{2263.1}{8} = 283 \text{ mm}$$

$$\therefore \quad a = 91.44 \text{ mm}$$

Provide 8 mm, weld of length 100 mm. Figure 9.6, with a = 100 mm, b = 110 mm, s = 8 mm, represents the design. Plate length may be kept

l = 120 + 100 = 220 mm so that a length of 120 mm is not welded.

* * *

Questions

1. Draw neat sketches and explain the design principles of the following welded connections for the transfer of shear only.
   (a) Double plate framed connection
   (b) Double angle framed connection
   (c) Unstiffened seat connection
   (d) Stiffened seat connection.
2. Explain the design principle of moment connection using tension plate and welds for transferring moment only.
3. An ISLB 350 beam is to be connected to a ISHB 250 @ 500 N/m to transfer a load of 100 kN. Design
   (a) Double plate framed connection
   (b) Double angle framed connection. Use welded connections.
4. An ISLB 400 transfers an end reaction of 130 kN to the flange of an ISHB 250 @ 500 N/m. Design
   (a) Unstiffened welded connection
   (b) Stiffened welded connection.
5. Design a welded connection to transfer a moment of 60 kN-m from an ISLB 500 to a column. Explain how the transfer of 120 kN shear can be made in the same connection.

10

DESIGN OF PLATE GIRDERS

When span and load increase, the available rolled section may not be sufficient, even after strengthening with cover plates. Such situations are common in the following:

1. Larger column free halls are required in the lower floor of a multistorey building.
2. In a workshop, where girders are required to carry crane beams.
3. In road or railway bridges.

In such situations one of the remedies is to go for a built up I-section with two flange plates connected to a web plate of required depth. The depth of such I-beams may vary from 1.5 m to 5.0 m.

This type of I-beams are known as **'Plate Girder'**. Figure 10.1 shows a typical plate girder.

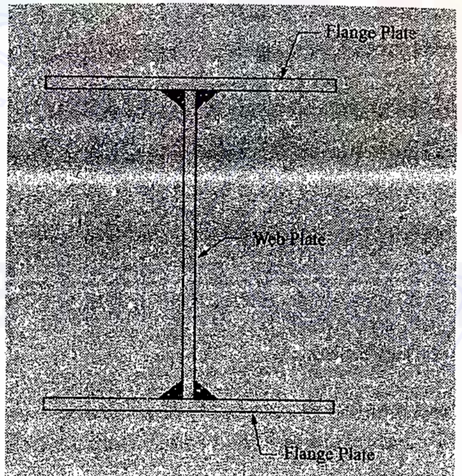

Figure 10.1

Before welding technology advanced, it was common practice to use riveted/bolted plate girders. Flange and web plates were connected to each other using angles and rivets/bolts. Many railway bridges of span 24 m to 46 m were built like this. This practice of using riveted/bolted plate girder is given up in 1960s. Nowadays only welded plate girders are built which are aesthetically good and at the same time light compared to riveted plate girders. Hence in this chapter design of only welded plate girder is presented.

10.1 ELEMENTS OF PLATE GIRDERS

The following are the elements of a typical plate girder [Ref. Fig. 10.2].

1. Web
2. Flanges
3. Stiffeners.

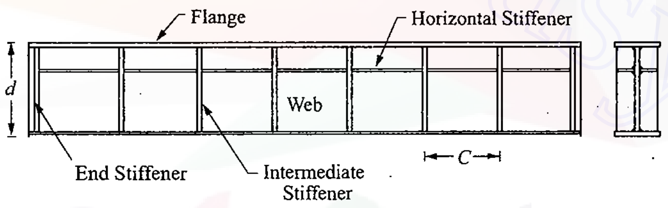

Figure 10.2

Webs of required depth and thickness are provided to:

(a) keep flange plates at required distances
(b) resist the shear in the beam.

Flanges of required width and thickness are provided to resist bending moment acting on the beam by developing compressive force in one flange and tensile force in another flange.

Stiffeners are provided to safeguard the web against local buckling failure. The stiffeners provided may be classified as:

(a) Transverse (vertical) stiffeners and
(b) Longitudinal (horizontal) stiffeners.

Transverse stiffeners are of two types:

(i) Bearing stiffener
(ii) Intermediate stiffener

End bearing stiffeners are provided to transfer the load from beam to the support. At the end certain portion of web of beam acts as a compression member and hence there is possibility of crushing of web. Hence web needs stiffeners to transfer the load to the support. If concentrated loads are acting on the plate girder (may be due to cross beams), intermediate bearing stiffeners are required.

To resist average shear stress, the thickness of web required is quite less. But use of thin webs may result into buckling due to shear. Hence when thin webs are used, intermediate transverse stiffeners are provided to improve buckling strength of web.

Many times longitudinal (horizontal) stiffeners are provided to increase the buckling strength of web. If only one longitudinal stiffeners is provided, it will be at a depth of 0.2 d from the compression flange where 'd' is the depth of web. If another longitudinal stiffener is to be provided it will at mid depth of web.

Web, flange and stiffeners are all plates. They are to be connected suitably by welding to form a single structural system i.e. plate girder. The plate girder has to resist shear force and bending moment acting on it. No plate should fail under any of the designed load. In this chapter, the design principle is explained to select suitable sizes of plates and also to design their welded connections. The procedure is illustrated with solved example.

10.2 SELF WEIGHT OF PLATE GIRDER

The following empirical formula may be used to assess the self weight of the beam:

$$w = \frac{W}{200} \text{ kN/m}$$

where w – factored self weight

and W – total factored load on the girder.

Considering this value of self weight and the other applied loads, moment M and shear force F to be considered for the design is found.

10.3 ECONOMICAL DEPTH

Assuming that the moment M is carried out by flanges only, the economical depth 'd' of girder may be found as given below:

$$M = f_y b_f t_f d$$

where b_f and t_f are the breadth and thickness of the flange.

$$\therefore \quad b_f \, t_f = \frac{M}{f_y \, d}$$

The gross sectional area of the girder

A is given by

$$A = 2\, b_f t_f + d\, t_w$$

$$= \frac{2M}{f_y d} + d\, t_w$$

Taking $\frac{d}{t_w} = k$, where k is assumed constant, then,

$$A = \frac{2M}{f_y d} + \frac{1}{k} d^2$$

For A to be minimum, the above expression is to be differential w.r.t. 'd' and equated to zero. Hence

$$0 = -\frac{2M}{f_y d^2} + \frac{1}{k} 2d$$

or

$$d^3 = \frac{Mk}{f_y}$$

or

$$d = \left[\frac{Mk}{f_y}\right]^{1/3}$$

The above expression may be used to get the idea about economical depth. To avoid labour cost of cutting or welding, the available plate size is used. It may be slightly less than the economical depth, since in deriving bending resistance, the contribution of web has been neglected.

In selecting the value of $k = \frac{d}{t_w}$, the following codal provisions will be useful:

I. If $\frac{d}{t_w} \le 67\epsilon$, it may be designed as ordinary beam as explained in chapter 7.

Note: $\epsilon = \left(\frac{250}{f_y}\right)^{1/2}$

II. **Minimum web thickness based on serviceability requirement** [clause 8.6.1.1 in IS 800]

(a) When transverse stiffeners are not provided, $\frac{d}{t_w} \le 200\epsilon_w$, web connected to flanges along both longitudinal edges.

(b) When the transverse stiffeners are provided;

1. when $3d \ge c \ge d$

$$\frac{d}{t_w} \le 200\epsilon$$

2. when $0.74d \le c < d$

$$\frac{c}{t_w} = 200\epsilon_w \quad \text{where} \quad \epsilon_w = \sqrt{\frac{250}{\text{yield stress of web}}}$$

3. when $c < d$,

$$\frac{d}{t_w} \le 270\epsilon_w$$

4. when $c > 3d$, the web shall be considered unstiffened.

(c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2 d from compression flange)

1. when $2.4\, d \ge c \ge d$

$$\frac{d}{t_w} \le 250\epsilon_w$$

2. when $0.74\, d \le c \le d$

$$\frac{c}{t_w} \le 250\epsilon_w$$

3. when $c < 0.74\, d$

$$\frac{d}{t_w} \le 340\epsilon_w$$

(d) When a second longitudinal stiffeners (located at neutral axis is provided)

$$\frac{d}{t_w} \le 400\epsilon_w.$$

III. **Minimum web thickness based on compression flange buckling requirement** (clause 8.6.1.2 in IS 800):

In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

(a) When transverse stiffeners are not provided

$$\frac{d}{t_w} \le 345\epsilon_f^2$$

where ϵ_f = yield stress ratio of flange

$$= \sqrt{\frac{250}{f_{yf}}}$$

(b) When transverse stiffeners are provided and

1. when $c \geq 1.5\,d$

$$\frac{d}{t_w} \leq 345 \in_f^2$$

2. when $c < 1.5\,d$,

$$\frac{d}{t_w} \leq 345 \in_f$$

From the above clauses, if Fe 415 (E250) steel is used ($\in = 1$), the following points may be observed.

(1) If $k = \frac{d}{t_w} \leq 67$, the plate girder may be designed as ordinary beam, i.e., without any stiffener (except end bearing stiffener). But such sections will be uneconomical

(2) If $k = \frac{d}{t_w}$ is between 67 to 200, it may be possible to have the plate girder without intermediate stiffeners. However designer has to check for the shear buckling of web. For k values upto 100–110, intermediate transverse stiffeners may not be required. But for larger k values consideration of web buckling may force to go for transverse stiffener.

(3) For k value upto 250, longitudinal stiffener is also required.

(4) k value should not be taken more than 345 to avoid compression flange failure.

In the past stiffener webs have been used. But present-day tendency is to avoid stiffeners to reduce fabrication cost and time. Hence it is preferable to go upto k = 100 to 110, so that real economical girder is obtained, provided it is safe in shear buckling of web without transverse stiffeners.

It may be noted that in all plate girders end transverse stiffeners are required to transfer the load to the support.

Another practical guide line for selecting the depth of plate girder is given below:

$$\frac{D}{L} = \frac{1}{15} \text{ to } \frac{1}{25} \text{ for girders in buildings}$$

$$= \frac{1}{12} \text{ to } \frac{1}{18} \text{ for highway bridges}$$

$$= \frac{1}{10} \text{ to } \frac{1}{15} \text{ for railway bridges.}$$

where D = depth of girder (including flange thicknesses)

and L = equivalent span of the girder.

10.4 SIZE OF FLANGES

Assuming moment is resisted by flanges only, and using material partial safety factor for a plastic section,

$$\frac{A_f \times f_y \times d}{1.1} = M$$

Hence area of flange A_f may be found. Select $9.4\in < t_f < 13.6\, b_f \in$ so that bending strength can be found by the formula for semi compact section as per the clause 8.2.1.2 in IS 800. Thus

$$b_f t_f = A_f$$

$$\text{i.e., } 13.6 \in t_f^2 = A_f.$$

Hence t_f is found. Then $b_f = \frac{A_f}{t_f}$.

10.5 SHEAR BUCKLING RESISTANCE OF WEB

For thin webs, it is necessary to check the shear resistance of web for buckling. IS 800-2007, clause 8.4.2 specify that this check is necessary when;

$$\frac{d}{t_w} > 67 \in \text{ for a web without stiffeners, and}$$

$$> 67 \in \sqrt{\frac{K_v}{5.35}} \text{ for a web with stiffeners}$$

where $K_v = 5.35$ when transverse stiffeners are provided at support

$$= 4.0 + \frac{5.35}{\left(\frac{c}{d}\right)^2} \quad \text{for} \quad \frac{c}{d} < 1.0$$

$$= 5.35 + \frac{4.0}{\left(\frac{c}{d}\right)^2} \quad \text{for} \quad \frac{c}{d} \geq 1.0$$

where c and d are the spacing of transverse stiffeners and depth of the web, respectively. The nominal shear strength $V_n = V_{cr}$ may be calculated by any one of the following two methods.

(a) Simple post-critical method

(b) Tension field method.

10.5.1 Simple Post-Critical Method

This method can be used for plate girders with or without transverse stiffeners. According to this method

$$V_n = V_{cr} = A_v \tau_b$$

where

τ_b = shear stress corresponding to web buckling which is to be determined as follows:

1. When $\lambda_w \leq 0.8$

$$t_b = \tau_b = \frac{f_{yw}}{\sqrt{3}} \cdot \quad f_{yw} = \text{yield stress of web material}$$

2. When $0.8 < \lambda_w < 1.2$

$$\tau_b = \left[1 - 0.8\left(\lambda_w - 0.8\right)\right]\frac{f_{yw}}{\sqrt{3}}$$

3. When $\lambda_w \geq 1.2$

$$\tau_b = \frac{f_{yw}}{\left(\sqrt{3}\,\lambda w^2\right)}$$

where λ_w = non-dimensional web slenderness ratio for shear buckling stress

$$= \left[\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}\right]^{1/2}$$

τ_{cr} = elastic critical shear stress of web

$$= \frac{K_v \pi^2 E}{12\left(1-\mu^2\right)\left(\frac{d}{t_w}\right)^2}$$

The Poisson's ratio μ for steel may be taken as 0.3.

10.5.2 Tension Field Method

This method of finding shear buckling strength of web may be used if end and intermediate transverse stiffeners are provided. It accounts for post buckling strength provided by the stiffeners. As the web begins to buckle, it loses ability to resist diagonal compression. At this stage, the transverse stiffeners and the flanges come into action to resist the diagonal compression. The vertical component of this compression is resisted by transverse stiffeners and horizontal component by the flange [Ref. Fig. 10.3]. The web resists only diagonal tension. Thus there is additional strength for resisting shear buckling. IS 800-2007 has accepted the following expression for computing shear resistance of web if end and intermediate stiffeners are provided and $\frac{c}{d} \geq 1.0$ [clause 8.4.2.2]

$$V_n = V_{tf}.$$

where

$$V_{tf} = [A_v \tau_b + 0.9\, w_{tf} t_w f_v \sin\phi] \leq V_p$$

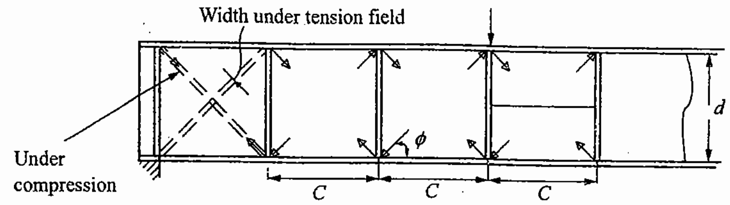

Figure 10.3

where

τ_b = buckling strength as obtained in simple post-critical method.

f_v = yield strength of the tension field obtained from

$= [f_{yw}^2 - 3\tau_b^2 + \psi^2]^{0.5} - \psi$

$\psi = 1.5\ \tau_b \sin 2\phi$

ϕ = inclination of the tension field

$\approx \tan^{-1}\frac{d}{1.5}$

w_{tf} = the width of the tension field

$= d\cos\varphi - (c - s_c - s_t)\sin\varphi$

s_c, s_t = anchorage lengths of tension field along the compression and tension flanges obtained from

$$s = \frac{2}{\sin\varphi}\left[\frac{M\,f_t}{f_{yw} t_w}\right]^{0.5} \leq c$$

where,

M_{fr} = reduced plastic moment capacity of the respective flange plate

$$= 0.25\, b_f t_f^2 f_{yf}\left[1 - \left\{\frac{N_f}{b_f t_f\, f_{yt} / \gamma_{mo}}\right\}^2\right]$$

N_f = Axial force in the flange.

10.6 END PANEL DESIGN

In a plate girder with transverse stiffeners, if the web is designed using tension field action, special care should be taken in the design of end panel. In IS 800-2007, clause 8.5 covers these provisions. The code

permits the design of end panels both by simple post buckling method and by tension field action with additional provisions given below.

10.6.1 If Simple Post Buckling Method is Used in the Design of End Panel

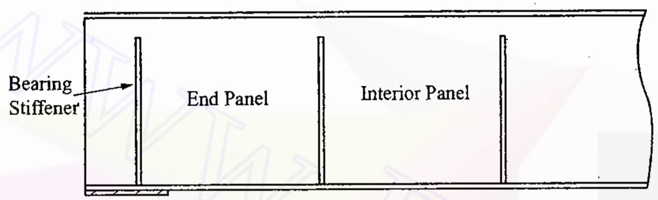

Figure 10.4

In this case the end panel along with the stiffeners (Fig. 10.4) should be checked as a beam spanning between the flanges to resist a shear force R_{tf} and a moment, M_{tf} due to tension field forces. Apart from this end stiffener should be capable of resisting the reaction plus a compressive force due to the moment equal to M_{tf}.

10.6.2 If End Panel is Designed Using Tension Field Action

If the end panel is also designed using tension field action, it should be provided with an end post consisting of a single or double stiffener (see Fig. 10.5 and Fig. 10.6).

Figure 10.5

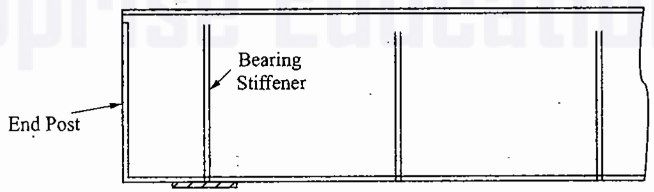

Figure 10.6

(a) *Single Stiffener.*

(i) The top of the end post should be rigidly connected to the flange using full strength weld.

(ii) The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to $\frac{2}{3}M_{tf}$ due to tension field forces.

(iii) The width and thickness of the end post are not to exceed the width and thickness of the flange.

(b) *Double Stiffener*

If double stiffeners as shown in Fig. 10.6 are provided, the end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and a moment M_{tf}.

10.7 ANCHOR FORCES

The resultant longitudinal shear R_{tf} and a moment M_{tf} from the anchor of tension field forces are to be evaluated as given below:

$$R_{tf} = \frac{H_q}{2} \text{ and } M_{tf} = \frac{H_q d}{10}$$

where,

$$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{0.5}$$

$$V_p = \frac{d\,t\,f_y}{\sqrt{3}}$$

d = web depth.

If the actual factored shear force, V (using tension field approach) is less than the shear strength, V_{tf}, then the values of H_q may be reduced by

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}}$$

where,

V_{tf} = the basic shear strength for the panel utilizing tension field action

V_{cr} = critical shear strength for the panel designed utilizing tension field action.

10.8 DESIGN OF CONNECTION BETWEEN FLANGE AND WEB PLATES

If 'V' is the shear force acting on the section, then shear stress at the junction is,

$$q_w = \frac{V}{bI_z}(a\,\bar{y})$$

$\therefore$ Shear force per unit length $= \frac{V}{I_z}(a\,\bar{y})$

If weld of throat thickness 't' is provided on both side, then strength of shop weld per unit length

$$= 2t\,\frac{f_w}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating the force to strength we get

$$\frac{V}{I_z}(a\,\bar{y}) = 2t\,\frac{f_w}{\sqrt{3}} \times \frac{1}{1.25}$$

Hence throat thickness of weld 't' can be found, from which size of the weld is obtained as $s = \frac{t}{0.7}$.

In finding shear stress, moment of inertia of flange alone may be considered i.e.

$$I_z = \frac{b_f\,d^3}{12}$$

If weld size comes out too small intermittent welding may be adopted.

10.9 DESIGN OF BEARING STIFFENERS

In case of rolled steel sections, the webs are so proportioned that it will safely carry load without buckling or crippling of the web. But in plate girders to achieve economy, webs are made thin. In such case the stiffeners are required at the ends to transfer the reaction safely. Stiffeners may be required, if the concentrated load are acting at some points in the girder. The stiffeners which transfer the load are known as bearing stiffeners.

Such stiffeners are placed in pairs, one on each side of web. As far as possible they should fit tightly between the flanges of girders and extend towards the edges of flange plates. They are tight fitted on compression flange side and welded to tensile flange.

Thickness of stiffeners is kept such that the outstand from the face of the web is not more than $20t_q\epsilon$ where ϵ_q is the thickness of stiffener.

When the outstand is between $14t_q\epsilon$ and $20t_q\epsilon$, the stiffener design is on the basis of a core section with an outstand of $14t_q\epsilon$.

These stiffeners should be checked for:

(a) Buckling resistance (clause 8.7.1.5)
(b) Bearing capacity (clause 8.7.5.2) and
(c) Torsional resistance (clause 8.7.9).

10.9.1 Buckling Resistance of Stiffeners

For this purpose the effective section is full area or core area of the stiffener together with an effective length of web on each side as shown in Fig. 10.7. It may be noted that some time (in case of end stiffener) there may not be web on one side of stiffener or, it may be less than 20 times web thickness. In such cases the available web size on that side is to be considered (Fig. 10.7.b)

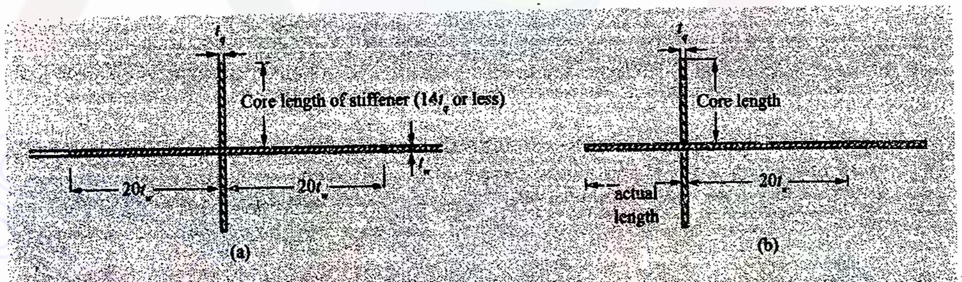

Figure 10.7

Buckling resistance may be found as explained below:

Find area, shown hatched in the Fig. 10.7.

Determine I_z.

Find $r = \sqrt{I/A}$

Take slenderness ratio $\lambda = 0.7\,\frac{L}{r}$.

Where L – length of stiffener.

From Table 9c in IS 800 (Table 6.4c in this book) find f_{cd}.

Then buckling resistance $= f_{cd} \times A >$ Applied load.

10.9.2 Check for Bearing Capacity of Stiffeners

The load carrying capacity of the stiffeners F_{psd} is given by

$$F_{psd} = \frac{A_q f_{yq}}{0.8\gamma_{mo}}$$

where

A_q = Area of stiffener in contact with flange

f_{yq} = yield stress of the stiffener.

In safe design $F_{psd} > F_x$

where F_x is external concentrated load or reaction.

10.9.3 Check for Torsional Resistance

After erecting ends of plate girder may have lateral restraint. But during transportation and erection, it may not have lateral restraint. Hence it is necessary to check for torsional resistance of the plate girder. Clause 8.7.9 in IS code specifies that

$$I_s \geq 0.34\,\alpha_s D^3 T_{cf}$$

where I_s = second moment of area of the stiffener section about central line of web.

$$\alpha_s = 0.006 \text{ for } \frac{L_{LT}}{r_y} \leq 50$$

$$= \frac{0.3}{(L_{LT}/r_y)} \quad \text{for} \quad 50 < \frac{L_{LT}}{r_y} = 100$$

$$= \frac{30}{(L_{LT}/r_y)^2} \quad \text{for} \quad \frac{L_{LT}}{r_y} > 100$$

L_{LT} = Effective length for lateral torsional buckling

D = Overall depth of beam at support

T_{cf} = Maximum thickness of compression flange in the span under consideration

r_y = radius of gyration of the beam about the minor axis.

10.10 WELD FOR END STIFFENERS

According to clause 8.7.2.6, these welds should be designed to transfer a shear equal to difference between reaction and the shear carried by web plus a shear between each component of the stiffener and the web of not less than

$$\frac{t_w^{\,2}}{5b_s} \text{ kN/mm}$$

where,

t_w = web thickness in mm

b_s = outstand width of the stiffener in mm.

The shear carried directly by the web may be found as explained below.

According to clause 8.7.3.1, the effective length of the web for evaluating the slenderness ratio is to be taken as

$$b = b_1 + n_1$$

where n_1 = dispersion of the load through the web at 45° to the level of the cross-section

= $D/2$, where D = overall depth of girder

Then obviously,

$$I = \frac{1}{12} d\,t_w^3$$

$$A = dt_w$$

$$\therefore \quad r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1}{12}\frac{dt^3}{dt}} = \frac{t}{\sqrt{12}}$$

$$\therefore \quad \lambda = \frac{0.7d}{r} = 0.7\sqrt{12}\,\frac{d}{t} = 2.42\,\frac{d}{t}$$

Using this value of slenderness ratio f_{cd} can be found from Table 9 (c) of code (Table 6.4 (c) in this book). Then buckling resistance = $bt\,f_{cd}$.

10.11 DESIGN OF INTERMEDIATE STIFFENERS

The design requirements for intermediate stiffeners are covered in clause No. 8.7.2 in IS code and they are presented below:

1. They may be provided on one or both sides of the web.
2. The spacing of stiffeners depends upon the web thickness (Fig. 10.3).

3. Outstand of stiffners should not be more than $20t_q\epsilon$. If outstand is more than $14t_q\epsilon$ and less than $20t_q\epsilon$, the core section width is to be considered $14\ t_q\epsilon$ only.
4. Minimum moment of inertia should satisfy the following requirement:

If $$\frac{c}{d} \geq \sqrt{2}, \quad I_s \geq 0.75 d\, t_w^2$$

and if $$\frac{c}{d} < \sqrt{2}, \quad I_s \geq \frac{1.5 d\, t_w^2}{c^2}$$

5. Stiffeners not subjected to external loads or moments should be checked for a stiffener force:

$$F_q = V - \frac{V_{cr}}{\gamma_{mo}} \leq F_{qd}$$

where

F_{qd} = design resistance of the intermediate stiffener

V = factored shear force adjacent to the stiffener

V_{cr} = shear buckling resistance of the web panel designed without using tension field action.

6. Intermediate stiffeners subjected to external loads should be designed as bearing stiffeners. In addition they should satisfy the following interaction formula:

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1$$

If $F_q < F_x$, then take $F_q - F_x = 0$.

Where

F_x = external load or reaction at the stiffener.

F_{xd} = design resistance of a load carrying stiffener corresponding to buckling about axis parallel to the web.

M_q = moment on the stiffener.

M_{yq} = yield moment capacity of the stiffener.

10.12 CONNECTION OF INTERMEDIATE STIFFENERS TO WEB

Intermediate transverse stiffeners not subjected to external loading should be connected to the web so as to withstand a shear between each component of the stiffener and the web of not less than

$$\frac{t_w^2}{5b_s} \text{ kN/mm.}$$

If it is subjected to load, then the design is similar to that for bearing stiffeners explained in Fig. 10.10.

10.13 PROCEDURE OF DESIGN OF PLATE GIRDER

1. Assuming self weight is equal to $\frac{W}{200}$, where W is total factored load, determine the factored shear force and moment.
2. Decide whether to use or not to use transverse stiffeners, and assume the value of k i.e., $\frac{d}{t_w}$. Determine economical depth as

$$d = \left[\frac{Mk}{f_y}\right]^{1/3}$$

select available plate around this depth.
3. Determine the area of flange required to resist moment. Proportion it so that $\frac{b_f}{t}$ satisfies requirements of plastic/compact/semi compact section.
4. Check the moment capacity of the girder.
5. Find shear resistance of the web using either simple post-critical method or by tension field method.
6. Design the weld connecting flange plate and web plate.
7. Design the end bearing stiffeners.
8. Design the connection of stiffener.
9. Design load carrying stiffeners, if required.
10. Design intermediate stiffeners, if required.

Web splice and flange splices may be designed as explained in chapter 4. It may be noted that to take care of varying moment flange thicknesses are varied at suitable length along the span.

Example 10.1

Design a welded plate girder of span 24 m to carry superimposed load of 35 kN/m. Avoid use of bearing and intermediate stiffeners. Use Fe 415 (E250) steel.

Solution:

1. *Moment and shear force:*

Span = 24 m.

Super-imposed load = 35 kN/m

∴ Factored load = 35 × 1.5 = 52.5 kN/m

Self weight $= \frac{52.5 \times 24}{200} = 6.3$ kN/m.

∴ Total factored load = 52.5 + 6.3 = 58.8 kN/m.

$$\therefore \quad \text{Maximum moment } M = \frac{wL^2}{8} = \frac{58.8 \times 24^2}{8}$$

$$= 4233.6 \text{ kN-m.}$$

Maximum shear force = End reaction

$$V = \frac{wL}{2} = \frac{58.8 \times 24}{2}$$

$$= 705.6 \text{ kN}$$

2. *Depth of web plate:*

If stiffeners are to be avoided,

$$k = \frac{d}{t_w} \le 67$$

$\therefore$ Economical depth of web

$$d = \sqrt[3]{\frac{Mk}{f_y}} = \left(\frac{4233.6 \times 10^6 \times 67}{250}\right)^{1/3}$$

$$= 1043 \text{ mm.}$$

Use 1000 mm plates.

$$t_w \ge \frac{1000}{67} \text{ i.e., } t_w \ge 14.92$$

Select $t_w = 16$ mm.

Thus web plate selected is 1000 mm × 16 mm.

3. *Selection of Flange:*

Neglecting the moment capacity of web, area of flange required is

$$\frac{A_f f_y d}{1.1} \ge M$$

$$\frac{A_f \times 250 \times 1000}{1.1} \ge 4233.6 \times 10^6$$

$$\therefore \quad A_f = 18628 \text{ mm}^2$$

To keep the flange in plastic category $\frac{b}{t_f} \le 8.4 \quad \therefore \frac{b_f}{2t_f} \le 8.4$

Assuming $b_f = 16.8\ t_f$

we get, $A_f = 16.8\ t_f t_f = 18628$

$$\therefore \quad t_f = 33.3 \text{ mm}$$

Select 40 mm plates. Width of plate required $= \frac{18628}{40} = 465.7$

Hence use 480 mm wide and 40 mm thick plates. Section selected is shown in Fig. 10.8.

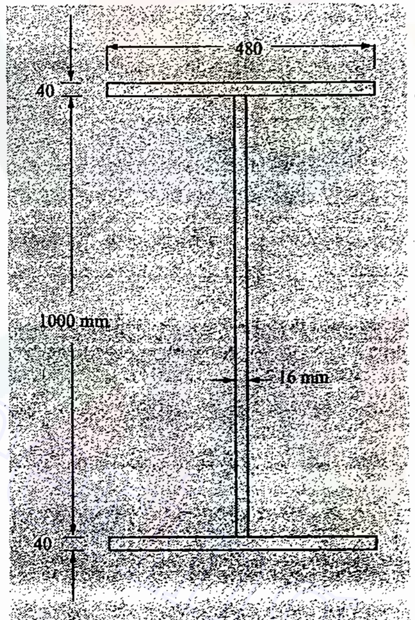

Figure 10.8

4. *Check for the moment capacity of the girder:*

Since it is assumed that only flanges resist the moment and flange is a plastic section, (clause 8.2.1.2).

$$M_d = \frac{Z_e f_y}{\gamma_{mo}} \le 1.2 \frac{Z_e f_y}{\gamma_{mo}}$$

$$\frac{Z_p f_y}{\gamma_{mo}} = \frac{40 \times 480 \times 250 \times 1040}{1.1} = 4538.182 \times 10^6 \text{ N-mm}$$

Now $$I_{zz} = 2\left[\frac{1}{12}\times480\times40^3 + 480\times40\times\left(\frac{1000+2\times40}{2}\right)^2\right]$$

$$= 2\times5601.28\times10^6 \text{ mm}^4$$

$$= \frac{1.2Z_e f_y}{1.1} = \frac{1.2\times I_{xx}}{y_{max}} = \frac{2\times5601.28\times10^6}{540} = 5657.72\times10^6 \text{ mm}^3$$

$$\therefore\quad M_d = 4538.182\times10^6 \text{ N-mm}$$

$$= 4538.182 \text{ kN-m} > M$$

Hence section is adequate.

5. *Shear resistance of web [clause 8.4]*

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\gamma_{mo}\sqrt{3}} = \frac{d t_w f_{yw}}{\gamma_{mo}\sqrt{3}}$$

$$\therefore\quad V_d = \frac{1000\times16\times250}{1.1\sqrt{3}} = 2099.455\times10^3 \text{ N}$$

$$= 2099.455 \text{ kN} > 705.6 \text{ kN}$$

Hence section is adequate.

No stiffeners are required.

6. *Check for end bearing:*

Bearing strength of web

$$F_w = (b_1 + n_2)t_w\frac{f_{yw}}{\gamma_{mo}}$$

Assuming that the width of support is 200 mm, minimum stiff bearing provided by support = 100 mm.

Dispersion length $n_2 = 2.5\times40 = 100$ mm

$$F_w = (100+100)\times16\times\frac{250}{1.1} = 727\times10^3 \text{ N} = 727 \text{ kN}.$$

$$> 705.6 \text{ kN}$$

Hence adequate.

End stiffener is also not required.

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7. *Design of weld connecting web plate and flange:*

Maximum shear force = 705.6 kN.

Shear stress in flange at the level of junction of web and flange

$$q = \frac{F}{bI}(a\bar{y})$$

$$= \frac{705.6\times10^3}{480\times2\times5601.28\times10^6}\left[480\times40\times\left(500+\frac{40}{2}\right)\right]$$

$$= 0.512 \text{ N/mm}^2$$

$\therefore$ Shear force per mm length in the junction

$$= 0.512\times480 = 245.76 \text{ N}$$

If s is the size of shop weld, throat thickness is $0.7s$. Providing weld on both sides of web strength per unit length

$$= 2\times0.75\times\frac{410}{\sqrt{3}}\times\frac{1}{1.25} = 265.1s$$

Equating it to shear force, we get

$$265.1\,s = 245.76$$

$$\therefore\quad s = 0.92 \text{ mm}.$$

But a minimum of 5 mm is to be provided since thickness of web is 16 mm. Intermittent welds may be provided [clause (0.5.5)].

$$\therefore\quad \%\text{ of weld length} = \frac{0.92}{5}\times100 = 18.4$$

Use 40 mm long welds with a gap of 160 mm which satisfies the clauses that

(a) Minimum weld length 40 mm
(b) Maximum unwelded length 12 × 16 = 192 mm

and also the required percentage welding.

Final Design:

Web: 1000 × 16 mm.

Flange: 480 × 40 mm.

No stiffeners are required.

Weld: 5 mm intermittent of length 40 mm and a gap of 160 mm.

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Example 10.2

Design the above plate girder using thin web and end stiffener, but avoid intermediate stiffeners.

Solution:

1. *Maximum moment* $M = 4236.6$ *kN-m* and Maximum shear force = 705.6 kN.
2. *Depth of web*

If $\dfrac{d}{t_w} > 200$, intermediate transverse stiffeners are to be provided.

If $\dfrac{d}{t_w} \leq 67$, end as well as intermediate transverse stiffeners are not required but thick webs are required. It is around $k = \dfrac{d}{t_w} = 100$, that thin webs with only end stiffeners can be used.

Try $k = \dfrac{d}{t_w} = 100$ in this case.

$$\text{Economical depth} = \sqrt[3]{\frac{Mk}{f_y}}$$

$$= \left[\frac{4233.6 \times 10^6 \times 100}{250}\right]^{1/3}$$

$$= 1192 \text{ mm.}$$

Provide 1200 mm wide plates.

$$\therefore \quad \frac{1200}{t_w} = 100$$

Hence $t_w = 12$ mm.

Use 1200 mm × 12 mm plates for web.

3. *Flange:*

Assuming only flanges resist moment, area of flange A_f required is given by,

$$\frac{A_f f_y d}{1.1} \geq 4233.6 \times 10^6$$

$$\frac{A_f \times 250 \times 1200}{1.1} \geq 4233.6 \times 10^6$$

or $$A_f \geq 15523 \text{ mm}^2.$$

To keep the flange in plastic category,

$$\frac{b}{t_f} \leq 8.4 \qquad \text{i.e.} \quad \frac{b_f}{2t_f} \leq 8.4$$

Assuming $b = 16.8\, t_f$, we get

$$A_f = 16.8\, t_f \times t_f = 15523$$

or $$t_f = 30.4 \text{ mm.}$$

$\therefore$ Select 36 mm plates.

$$b = \frac{15523}{36} = 431 \text{ mm}$$

Use 440 mm wide and 36 mm thick plates. Preliminary section selected is as shown in Fig. 10.9.

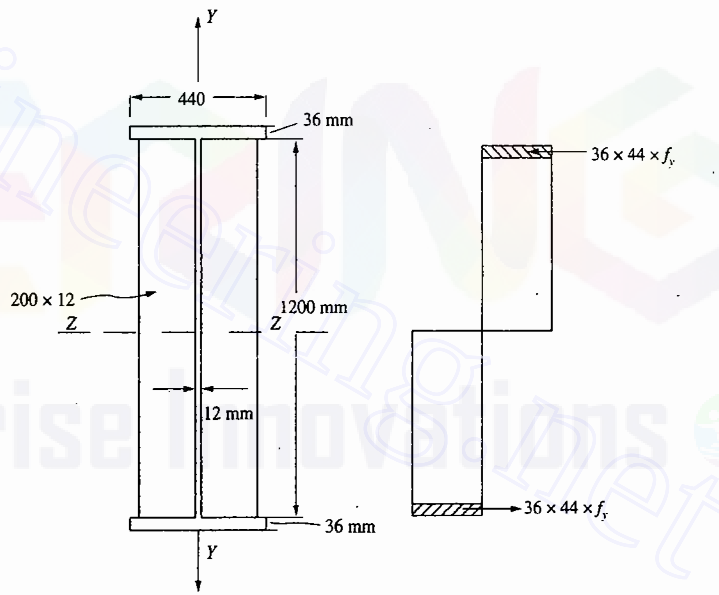

Figure 10.9

4. *Check for moment capacity:*

Since it is plastic section, $M_d = \frac{Z_p f_y}{1.1} \le 1.2 \frac{Z_e f_y}{1.1}$

$$\frac{Z_p f_y}{1.1} = \frac{36 \times 440 \times 250}{1.1} = 4449.6 \times 18 \text{ N-mm} = 4449.6 \text{ kN-m}$$

Assuming only flanges resist moment

$$I_{zz} = 2\left[\frac{1}{12} \times 440 \times 36^3 + 440 \times 36 \times \left(600 + \frac{36}{2}\right)^2\right]$$

$$= 1.21028 \times 10^{10}$$

$$\therefore \quad Z_e = \frac{I_{zz}}{y_{max}} = \frac{1.21028 \times 10^{10}}{636} = 19.0296 \times 10^6 \text{ mm}^3$$

$$\frac{1.2 \times Z_e f_y}{\gamma_{mo}} = \frac{1.2 \times 19.0296 \times 10^6 \times 250}{1.1} = 5189.9 \times 10^6 \text{ N-mm}^4$$

Hence adequate. $\quad \therefore \quad M_d = 4449.6 \text{ kN-m} > M$

5. *Shear resistance of web:*

Since transverse stiffeners are to be provided only at support, $K_v = 5.35$ and in this case

$$\frac{d}{t_w} > 67$$

Hence resistance to shear buckling should be verified. Consider simple post critical method (clause 8.4.2.2).

$$\tau_{cr} = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1200}{12}\right)^2}$$

$$= 96.7 \text{ N/mm}^2$$

$$\therefore \quad \lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3} \times 96.7}} = 1.22.$$

Since it is more than 1.2,

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda w^2} = \frac{250}{\sqrt{3} \times 1.22^2} = 96.97 \text{ N/mm}^2$$

$$\therefore \quad V_{cr} = d_{tw}\, \tau_b$$

$$= 1200 \times 12 \times 96.97 = 1396.44 \times 10^3 \text{ N}$$

$$= 1396.44 \text{ kN.}$$

$$\therefore \quad V_d = \frac{V_n}{\gamma_{mo}} = \frac{V_{cr}}{\gamma_{mo}} = \frac{1396.44}{1.1}$$

$$= 1269.49 \text{ kN} > 705.6 \text{ kN.}$$

Hence shear strength is adequate.

6. *Local capacity of the web (clause 8.7.4):*

As per the clause 8.7.4, local capacity of web

$$F_w = \frac{(b_1 + n_2)\, t_w\, f_{yw}}{\gamma_{mo}}$$

Taking $\quad b_1 = 0$

$$n_2 = 2 \times 2.5 \times 36 = 180 \text{ mm.}$$

$$\therefore \quad F_w = \frac{[0 + 180] \times 12 \times 250}{1.1} = 490.909 \times 10^3 \text{ N} = 545.45 \text{ kN}$$

$$< 705.6 \text{ kN}$$

Hence end stiffeners should be provided.

7. *Design of end stiffeners:*

Outstand of flange $= \frac{440 - 12}{2} = 214$ mm

Try a pair of 200×12 mm flats.

Now $14\, t_f = 14 \times 12 = 168$ mm.

$\therefore$ Core area of stiffener on each side $= 168 \times 12$ mm^2.

Figure 10 shows the core area of stiffener along with effective area of web ($20\, t_w$), assuming web area is available on only one side.

Area for buckling resistance $= (2 \times 168 + 20 \times 12) \times 12$

$$= 6912 \text{ mm}^2.$$

$$I_x = \frac{1}{12} \times 12(168 + 168 + 12)^3 + \frac{1}{12}(20 \times 12) \times 12^3$$

$$= 42.179 \times 10^6 \text{ mm}^4.$$

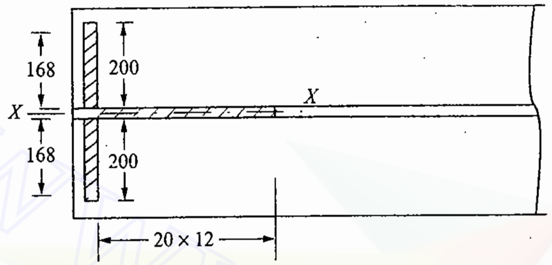

Figure 10.10

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.175 \times 10^6}{6912}}$$

$$= 78.12 \text{ mm}$$

$$\therefore \quad \lambda = 0.7 \times \frac{1200}{78.12} = 10.75$$

From Table 9c in IS 800 (Table 6.4c in this book)

$$f_{cd} = 226.5 \text{ N/mm}^2$$

$\therefore$ Buckling resistance $= 226.5 \times 6912$

$$= 1565.5 \times 10^3 \text{ N}$$

$$= 1565.5 \text{ kN} > 705.6 \text{ kN.}$$

Hence the stiffener is safe.

Check for bearing capacity of stiffener (clause 8.7.5.2):

$$F_{psd} = \frac{A_q f_{yq}}{0.8 \gamma_{mo}}$$

where

A_q = Area of stiffener in contact with flange

$= 2 \times 200 \times 12 = 4800 \text{ mm}^2$.

$$\therefore \quad F_{psd} = \frac{4800 \times 250}{0.8 \times 1.1} = 1363.6 \times 10^3 \text{ N}$$

$$= 1363.6 \text{ kN} > 705.6 \text{ kN.}$$

Hence the stiffener is adequate.

Check for torsional restraint provided by stiffener:

In actual use there is lot of torsional restraint, but during transport, restraint is by stiffeners only. As per clause 8.7.4,

$$I_s \geq 0.34\, \alpha_s D^3 T_{cf}$$

For girder

$$I_y = 2 \times \frac{1}{12} \times 36 \times 440^3 + \frac{1}{12} \times 1200 \times 12^3$$

$$= 511.277 \times 10^6 \text{ mm}^4$$

$$A = 2 \times 36 \times 440 + 1200 \times 12 = 46080 \text{ mm}^2$$

$$\therefore \quad r_y = \sqrt{\frac{511.277 \times 10^6}{46080}} = 105.3 \text{ mm}$$

$$\therefore \quad \lambda = \frac{24 \times 1000}{105.3} = 227.92$$

$$\therefore \quad a_s = \frac{30}{\lambda^2} = \frac{30}{227.92^2} = 5.779 \times 10^{-4}$$

$$I_s \geq 0.34\, \alpha_s D^2 t_{cf}$$

$$\geq 0.34 \times 5.775 \times 10^{-4} \times (1272)^3 \times 36$$

$$\geq 14.548 \times 10^6$$

Now, I_s = Second moment of area of stiffener about *x-x* axis (Fig. 10.10)

$$= \frac{1}{12} \times 12 \times (200 + 12 + 200)^3 + \frac{1}{12} \times 12 \times 12^3$$

$$= 69.933 \times 10^6 > 14.558 \times 10^6.$$

Hence torsional restraint is sufficient.

8. *Weld connecting web and flange:*

Shear force $V = 705.6$ kN $= 705.6 \times 10^3$ N

$I_z = 1.21028 \times 10^{10}$ mm^4 [As found in (4) above (A)]

$$\therefore \quad \text{Shear stress} = \frac{V}{bI_z}(a\bar{y})$$

$$\therefore \quad \text{Shear force per unit length} = \frac{V}{I_z}(a\bar{y}) = \frac{705.6 \times 10^3}{1.21028 \times 10^{10}}(36 \times 440 \times 618) = 570.71 \text{ N/mm}^2$$

If size of field weld is s and if it is provided on both side of web, strength of weld per unit length

$$= 2(0.7s)\times 1\times \frac{f_u}{\sqrt{3}}\times \frac{1}{1.25}$$

$$= 2\times 0.70\times s\times \frac{410}{\sqrt{3}}\times \frac{1}{1.25}$$

$$= 265.1\ s\ \text{N/mm.}$$

Equating it to shear force we get,

$$265.1\,s = 570.71$$

$$\therefore \quad s = 2.153\ \text{mm.}$$

For 12 mm plates, minimum size of the weld to be used is 5 mm.

$\therefore$ Percentage of welded length required

$$= \frac{2.153}{5}\times 100 = 43.06$$

Provide 50% weld

Minimum length of intermittent welds = 40 mm.

$\therefore$ Provide 40 mm length with 5 mm weld on both sides and then give a gap of 40 mm.

9. *Weld connecting stiffener and web:*

Shear carried by web directly:

$$\lambda = 2.42\frac{d}{t} = 2.42\frac{1200}{12} = 242$$

$$n_1 = \frac{D}{2} = \frac{1200+2\times 36}{2} = 636\ \text{mm}$$

From Table 9 (c) in IS:800,

$$f_{cd} = 26.2 - \frac{2}{10}(26.2 - 24.3)$$

$$= 25.82\ \text{N/mm}^2$$

Assuming stiff bearing, $b_1 = 0$

Area of web resisting shear $= bt_w = 636\times 12$

$\therefore$ Load directly transferred by web $= 636\times 12\times 25.82 = 197.08\times 10^3$ N

$$= 197.08\ \text{kN}$$

$\therefore$ Shear to be transferred through weld = Reaction − shear directly transferred by web

$$= 705.6 - 197.08 = 508.52\ \text{kN}$$

Length of weld $= 1200 - 2\times 12 = 1176$ mm

Shear to be transferred

$$= \frac{508.52}{1176} = 0.432\ \text{kN/mm}$$

To this additional shear of

$$= \frac{t_w^2}{5b_s} \quad \text{is to be added}$$

$$= \frac{12\times 12}{5\times 200} = 0.144\ \text{kN/mm}$$

$\therefore$ Total design shear for weld $= 0.432 + 0.144 = 0.576$ kN/mm

$$= 576\ \text{N/mm.}$$

If 's' is the field weld size and weld is provided on both sides, strength of weld

$$= 2\times 0.7s\times \frac{410}{\sqrt{3}}\times \frac{1}{1.5} = 220.9s\ \text{N/mm}$$

Equating it to design shear we get $220.9s = 556$

$$\therefore \quad s = 2.62\ \text{mm.}$$

For 12 mm plates minimum weld size is 5 mm.

$$\therefore \quad \text{Percentage welding} = \frac{2.62}{5}\times 100 = 52.4$$

Provide 55% weld.

Provide 5 mm size intermittent welds, for a length of 55 mm and a gap of 45 mm. Welding should be on both sides.

Thus the final design is:

Web: 1200 mm × 12 mm.

Flange: 440 × 36 mm.

End stiffeners: 200 × 12 mm.

Weld connecting flange and web: 40 mm with a gap of 140 mm, size 5 mm, on both sides. Weld connecting stiffener and web: 55 mm and a gap of 45 mm, size 5 mm, on both sides.

Example 10.3

Design the plate girder given in example 10.1 using intermediate stiffeners.

Solution:

1. *Moment and shear*

Maximum moment = 4233.6 kN-m

Maximum shear force = 705.6 kN

2. *Depth of web*

If stiffener spacing 'c' is between 'd' and '$3d$' where 'd' is depth of web, then serviceability requirement is $k = \frac{d}{t_w} \le 200$

Taking $k = \frac{d}{t_w} = 190$, we get economical depth as

$$d = \left[\frac{Mk}{f_y}\right]^{1/3}$$

$$= \left[\frac{4233.6 \times 10^6 \times 190}{250}\right]^{1/3}$$

$$= 1476 \text{ mm}$$

Use 1500 mm wide plates.

$$t_w = \frac{1500}{190} = 7.89 \text{ mm}$$

∴ Use 1500 mm wide, 8 mm thick plates.

Provide stiffeners at every 2 m interval ($3d \ge c \ge d$).

3. *Flange*

Assuming flange alone resists the moment,

$$\frac{A_f \times f_y \times d}{1.1} \ge M$$

$$\frac{A_f \times 250 \times 1500}{1.1} \ge 4233.6 \times 10^6$$

$$\therefore \quad A_f \ge 12418 \text{ mm}^2.$$

To keep flange in plastic class,

$b < 8.4 f_b$ i.e., $\quad b_f/2 \le 8.4\, t_f$

Taking $\quad b_f = 16.8\, t_f$,

we get, $\quad 16.8\, t_f \times t_f \ge 12418$

i.e., $\quad t_f \ge 27.1$

Provide 32 mm plates.

$$\therefore \quad b_f = \frac{12418}{32} = 388 \text{ mm}$$

Use 400 mm wide, 32 mm thick plates.

The trial section selected is shown in Fig. 10.11.

4. *Check for shear buckling:*

Using simple post critical method (clause 8.4.2.2 a)

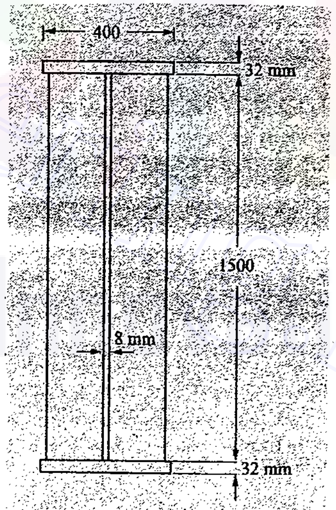

Figure 10.11

For $\frac{c}{d} \geq 1.0$

$$K_v = 5.35 + \frac{4}{\left(\frac{c}{d}\right)^2} = 5.35 + \frac{4}{\left(\frac{2000}{1500}\right)^2} = 7.6$$

$$\tau_{cr} = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} = \frac{7.6 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1500}{8}\right)^2}$$

$$= 39.08$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3} \times 39.08}} = 1.92$$

Since $\gamma_w > 1.2$,

$$\tau_b = \frac{f_{yw}}{\left(\sqrt{3}\,\lambda_w^2\right)} = \frac{250}{\sqrt{3} \times 1.92^2} = 39.15 \text{ N/mm}^2$$

$$\therefore \quad V_n = V_{cr} = A_v \tau_b$$

$$= 1500 \times 8 \times 39.15$$

$$= 469.800 \times 10^3 \text{ N} = 469.8 \text{ kN} < 705.6 \text{ kN}.$$

Hence intermediate stiffeners are to be used to improve buckling strength of the slender web and shear capacity of end panel should be checked.

5. *Check for the end panel*

Since it is going to be stiffened web panel, it should be checked as per clause 8.5.3 of IS 800.

$$V_p = \frac{d\,t\,f_y}{\sqrt{3}} = \frac{1500 \times 8 \times 250}{\sqrt{3}} = 1732.05 \times 10^3 \text{ N}$$

$$= 1732.05 \text{ kN}$$

$$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{0.5}$$

$$= 1.25 \times 1732.05 \left(1 - \frac{469.8}{1732.05}\right)^{0.5}$$

$$= 1848.26 \text{ kN}$$

$$\therefore \quad R_{tf} = \frac{H_q}{2} = 924.13 \text{ kN}$$

$$M_{tf} = \frac{H_q d}{10} = \frac{1848.26 \times 1500}{10} = 277239 \text{ kN-mm}$$

$$= 277.239 \text{ kN-m}.$$

The end panel is to be checked as a beam spanning between the flanges to resist R_{tf} and M_{tf}.

Area resisting shear = $t_w d = 8 \times 1500 = 12000$ mm^2

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}} = \frac{12000 \times 250}{\sqrt{3} \times 1.1} = 1574.59 \times 10^3 \text{ N}$$

$$= 1574.59 \text{ kN} > 924.13 \text{ kN}.$$

End panel can safely carry the shear due to the anchoring forces.

$$I = \frac{1}{12} t_w c^3 = \frac{1}{12} \times 8 \times 2000^3 = 5333.3 \times 10^6 \text{ mm}^4$$

$$y_{max} = \frac{c}{2} = \frac{2000}{2} = 1000 \text{ mm}$$

$$M_q = \frac{I}{y_{max}} \times \frac{f_y}{\gamma_{mo}} = \frac{5333.3 \times 10^6}{1000} \times \frac{250}{1.1}$$

$$= 1212.12 \times 10^6 \text{ N-mm}$$

$$= 1212.12 \text{ kN-m} > M_{tf}$$

Hence the end panel can safely carry the bending moment due to anchor forces.

6. *Design of end stiffeners*

Reaction at end = 705.6 kN.

Compressive force due to the moment M_{tf}

$$= \frac{M_{tf}}{c} = \frac{277.239 \times 10^6}{2000} = 138.62 \times 10^3 \text{ N}$$

$$= 138.62 \text{ kN}$$

$$\therefore \quad \text{Total compression} = 705.6 + 138.62$$

$$= 844.22 \text{ kN}.$$

Strength of the stiffener (clause 8.7.5.2):

$$F_{psd} = \frac{A_q f_{yq}}{0.8\gamma_{mo}} = \frac{A_q \times 250}{0.8 \times 1.1}$$

Equating strength to the force to be resisted we get,

$$\frac{A_q \times 250}{0.8 \times 1.1} = 844.22 \times 10^3$$

$$\therefore \quad A_q = 2972 \text{ mm}^2$$

Provide 200 mm wide, 10 mm thick flats on either side of web. Then *Aq* provided

$$= 2 \times 200 \times 10 = 4000 \text{ mm}^2 > 2972 \text{ mm}^2.$$

Check for outstand:

It should not be more than $20\, t_q = 20 \times 10 = 200$ mm

This requirement is satisfied.

Since it is more than 14×10, the core section is based on the width $14 \times 10 = 140$ mm^2

$\therefore$ Core area of each stiffener $= 140 \times 10 = 1400$ mm^2.

Buckling check for stiffeners

The effective area acting as column is as shown is Fig. 10.11(b)

Considering stiffeners only,

$$I_x = \frac{1}{12} \times 10 \times (140 + 8 \times 140)^3 + \frac{1}{12} + 160 \times 8^3 = 19.913 \times 10^6 \text{ mm}^4$$

Effective area $= 2 \times 140 \times 10 + 160 \times 8 = 4080$ mm^2

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{19.913 \times 10^6}{4080}} = 69.86 \text{ mm}^2$$

$kL_c = 0.7 \times d = 0.7 \times 1500 = 1050$ mm.

$$\lambda = \frac{k \cdot L_c}{r} = \frac{1050}{69.86} = 15.03$$

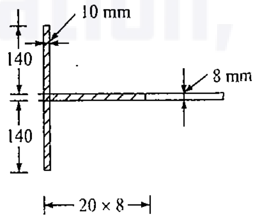

Figure 10.11 (b)

$\therefore$ From Table 9 (c) of IS 800 (Table 6.4 in this book)

$$f_{cd} = 227 - \frac{5.03}{10}(227 - 224) = 225.5 \text{ N/mm}^2.$$

Effective area = 4080 mm^2

$\therefore$ Buckling resistance of stiffener $= 4080 \times 225.5 = 920040$ N

$= 920.04$ kN

This is more than 844.22 kN.

Hence stiffener is adequate.

Checking stiffener for load bearing (clause 8.7.4):

Taking stiff bearing $\quad b_1 = 0$

$$n_2 = 2.5 \times t_f = 2.5 \times 32 = 80 \text{ mm}.$$

Local capacity of web

$$F_w = \frac{(b_1 + n_2) t_w f_{yw}}{\gamma_{mo}} = \frac{(0 + 80) \times 8 \times 250}{1.1}$$

$$= 145454 \text{ N}$$

$$= 145.454 \text{ kN}$$

$\therefore$ The stiffener is to be designed for a force $= 844.22 - 145.454$

$$= 698.766 \text{ kN}.$$

Area of stiffener alone $= 2 \times 200 \times 10 = 4000$ mm^2.

$\therefore$ Bearing capacity of stiffener alone

$$= \frac{250 \times 4000}{1.1} = 909.09 \times 10^3 \text{ N}$$

$$= 909.09 \text{ kN} > 698.766 \text{ kN}.$$

Hence the stiffener is safe.

Thus end stiffeners of size 200 mm × 10 mm are adequate.

7. *Design of intermediate stiffeners*

As the shear force goes on reducing towards mid span, the first stiffener from end is critical, Since first intermediate stiffener is at $c = 2$ m from end,

shear on this stiffener $= R - 2w$

$$= 705.6 - 2 \times 58.8 = 588 \text{ kN}.$$

In this case $\quad c = 2000$ mm

$\quad d = 1500$ mm.

$$\therefore \quad \frac{c}{d} = \frac{2000}{1500} = 1.33 \quad < \sqrt{2}$$

Hence minimum $I_s = \dfrac{1.5 d^3 t_w^3}{c^2}$

$$= \frac{1.5 \times 1500^3 \times 8^3}{2000^2} = 648000 \text{ mm}^4$$

Try intermediate stiffeners of size 120 × 10 mm on each side. This is not violating outstand clause ($< 20 t_q$)

$$I_s = \frac{1}{12} \times 10 \times (120 + 8 + 120)^3 - \frac{1}{12} \times 10 \times 8^3$$

$$= 12.71 \times 10^6 \text{ mm}^4 > I_s \text{ required.}$$

Hence adequate.

Check for buckling

Shear buckling resistance of the web alone

$$V_{cr} = 469.8 \text{ kN (Eqn. (A) on page 328)}$$

∴ Shear strength of stiffeners alone required $= \dfrac{V - V_{cr}}{\gamma_{mo}} = \dfrac{588 - 469.8}{1.1} = 107.45$ kN

Buckling resistance of intermediate stiffener (clause 8.7.15):

Considering $20 \times t_w = 20 \times 8 = 160$ mm width of web on both side along with stiffeners [Ref. Fig. 10.12].

$$I_x = I_s + 2 \times \frac{1}{12} \times 160 \times 8^3$$

$$= 12.71 \times 10^6 + 13653$$

$$= 12.73 \times 10^6 \text{ mm}^4.$$

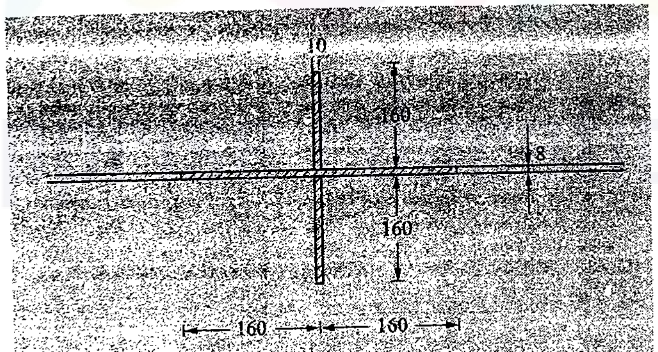

Figure 10.12

Area $= 2 \times 120 \times 10 + 2 \times 160 \times 8 = 4960$ mm^2

$$\therefore \quad r = \sqrt{\frac{12.73 \times 10^6}{4960}} = 50.66$$

$kL = 0.7 \times 1500 = 1050$ mm.

$$\lambda = \frac{1050}{50.66} = 20.73$$

From Table 9c in IS 800 (Table 6.4c in this book)

$$f_{cd} = 224 \text{ N/mm}^2.$$

∴ Buckling resistance $= 224 \times 4960 = 1111 \times 10^3$ N = 1111 kN

This is more than required resistance of 107.45 kN. Hence the stiffener is safe.

∴ Design of welds are to be carried out similar to those in example 10.2.

10.14 SUMMARY

Economical depth of plate girder is given by

$$d = \left(\frac{MK}{f_y}\right)^{1/3} \quad \text{where} \quad K = \frac{d}{t_w}$$

If $K \leq 67 \in$, it is thick web design. It does not require any stiffener.

If K is around 100, it is economical and it may not require intermediate stiffener. However end stiffeners are required.

For larger value of K, intermediate stiffeners are required. In such cases end panel should be checked for local failures by simple post buckling method or by considering tension field method.

As 'K' increases thickness of web reduces but depth increases. The thickness of flange also reduces. There will be overall economy in the girder section requirement. However the need for stiffener arises. Hence overall cost should be estimated considering the weight of girder along with stiffeners and also by adding added fabrication cost.

In general economical girder can be obtained by using a thin section with only end stiffeners. This is possible when K is around 100.

* * *

Questions

1. What is plate girder? Where it is used? Explain its various components with sketches.
2. Derive the expression for the economical depth of a plate girder. Assume moment is resisted by flanges only.
3. A plate girder is subjected to a maximum factored moment of 4000 kN-m and a factored shear force of 600 kN. Find the preliminary sections for the following conditions:
   (a) Girder without any stiffener
   (b) Girder with end stiffeners only
   (c) Girder with end as well as intermediate transverse stiffeners.
4. Explain the tension field action of thin web plates.
5. A plate girder with Fe 415 steel plates is having 12 mm × 1500 mm web plates and 56 mm × 500 mm flange plates. Determine the
   (a) Design flexural strength, if the compression flange is supported laterally.
   (b) Design strength in shear, if no intermediate stiffeners are used.
   (c) Design shear strength, if stiffeners are provided at every 2 m interval.
6. Design a simply supported plate girder of span 15 m carrying a factored udl of 48 kN/m, using only end stiffeners. Assume compression flange is laterally supported.
7. A plate girder is made with Fe 415 steel plates. The web plate is of the size 1200 × 12 mm and flange of size 440 × 36 mm. Check the adequacy of a pair of stiffeners of size 200 × 12 mm.

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11

DESIGN OF GANTRY GIRDERS

The travelling over-head cranes are commonly used in factories and workshops to lift and move heavy materials and assembled parts from one point to other. The crane system consists of a bridge over which a crab (trolley), hoist and cabin which houses the controls and operator move [Ref. Fig. 11.1]. The crane bridge (girder) itself is provided with wheels to move over the rails provided over gantry girder. Thus gantry girder supports crane girder. The gantry girder is supported on the columns with bracket. Figure 11.1 shows the typical system of gantry girder and crane.

The size of crab, wheel spacing etc., depend upon the capacity of the crane. These details are standardised and the manufactures supply them. The cranes may be operated manually or by electrically.

11.1 LOADS

The following imposed loads should be considered in the design.

1. Vertical loads from crane.
2. Impact loads from crane.
3. Longitudinal horizontal force along the crane rail.
4. Lateral thrust (surge) across the crane rail.

In calculating the above forces crane should be positioned such that it gives maximum forces in the girder. In case of tandum operation with more cranes, the positioning of all cranes for getting maximum forces in the girder should be considered. The impact loads to be considered are as presented in the Table 11.1.

Wherever necessary the design should be checked for earthquake forces and secondary effects such as handling, erection, temperature effects and fatigue.

11.2 POSITION OF MOVING LOAD FOR MAXIMUM EFFECTS

The following points studied in structural analysis should be noted for positioning moving loads for maximum effects.

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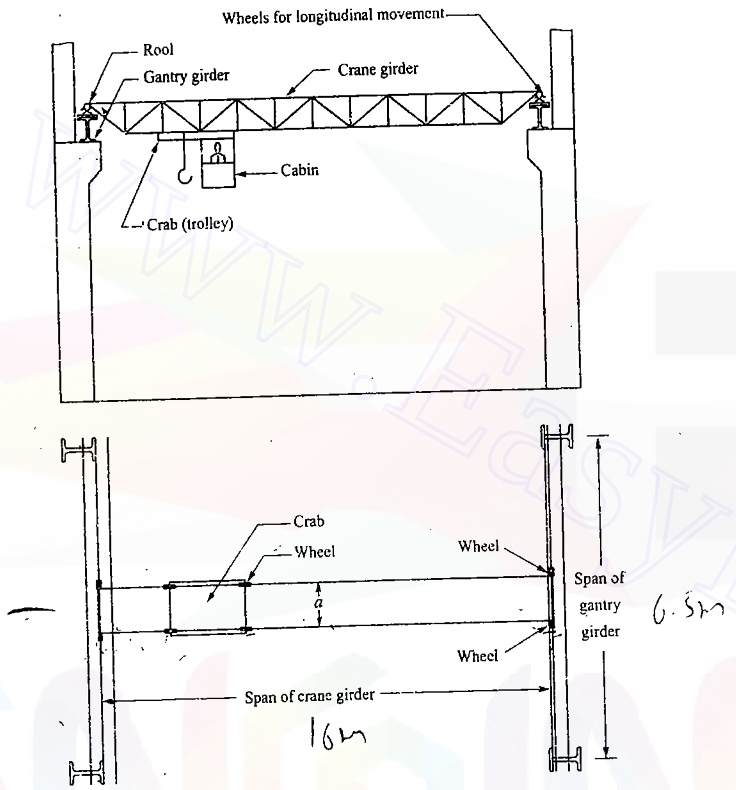

Figure 11.1

Table 11.1 Additional impact loads

Types of Loads	**Impact Allowance**
1. Vertical Loads	
(a) For electric operated cranes	25% of maximum static wheel loads.
(b) For hand operated cranes	10% of maximum wheel loads.
2. Horizontal Forces Transverse to Rails	
(a) For electric operated cranes	10% of weight of trolley and weight lifted.
(b) For hand operated cranes	5% of weight of trolley and weight lifted.
3. Horizontal Force Along the Rails	5% of the static wheel loads.

1. Position of Crane Hook for Maximum Vertical Load on Gantry Girder

The maximum vertical load on gantry girder is the maximum reaction of crane girder. To get this, crab should be placed as close to gantry girder as possible.

Referring to Fig. 11.2,

if, L_c – span of crane girder

L_1 – minimum approach of crane hook (distance between c.g. of gantry girder and trolley)

W – weight of trolley plus the weight lifted

w – weight of crane girder per unit length

$$R_A = \frac{1}{L_c}\left(\frac{wL_c^2}{2} + W(L_c - L_1)\right) \qquad \text{...(11.1)}$$

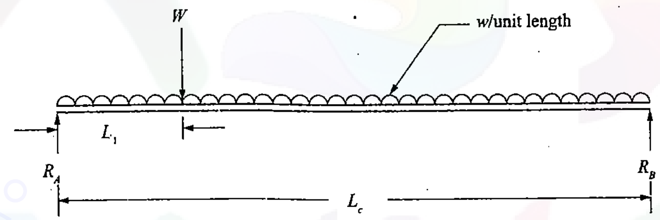

Figure 11.2

2. Position of Crane Wheels for Maximum Effects on Gantry Girder

The R_{max} calculated in equation 11.1, gets equally distributed between the two wheels of crane girder. Hence the moving load on gantry girder is wheel loads of $W = \frac{1}{2}R$ acting at a distance of '*a*' where '*a*' is the distance between the crane wheels. The position of these loads for maximum moment is shown in Fig. 11.3. Note that the maximum moment occurs under a load when the line of action of that load and the c.g. of the loads are at equal distance from the centre of span.

Maximum shear force in gantry girder occurs when both wheel loads are on the girder and are positioned as close to support as possible. Figure 11.4 shows this position.

Note: If two cranes are operated on a gantry girder, their position for maximum effect should be considered suitably.

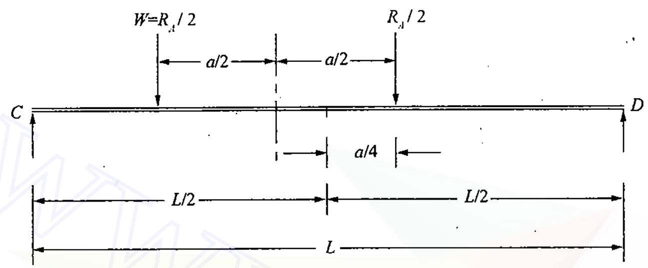

Figure 11.3

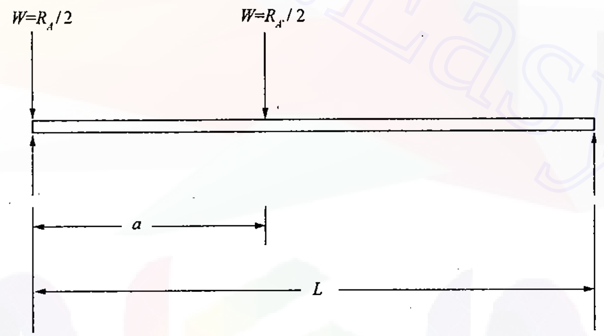

Figure 11.4

11.3 PROFILE OF GANTRY GIRDER

To resist heavy moment, I sections are required. They may be large rolled steel sections or even built up plate girders. Apart from bending moment and shear, these girders are subjected to longitudinal and lateral forces on compression flange. Hence compression flange needs additional strengthening. This is achieved by connecting a channel section on compression flange of I-section. Figure 11.5 shows typical gantry girder section, very commonly used. If stronger sections are required the rolled I-sections strengthened with plates may be used as shown in IS Hand Book SP(6). [Ref. Figs. 11.5b & c].

11.4 LIMITATION ON VERTICAL DEFLECTION

To avoid wear and tare of rails and excessive vibration code restricts vertical deflection as shown in Table 11.2.

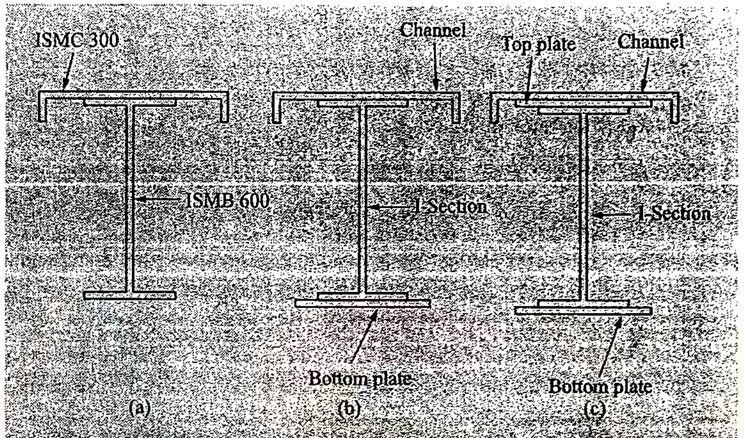

Figure 11.5

Table 11.2 Deflection limits on gantry girders
[Extract from Table 6 in IS 800-2007]

Category	Maximum Deflection
Vertical Deflection	
(a) Manually operated cranes	Span/500
(b) Electrically operated cranes upto 500 kN	Span/750
(c) Electrically operated cranes over 500 kN	Span/1000
Lateral Deflection	
Relative displacement between rails supporting 10 mm or crane	Span/400

11.5 DESIGN PROCEDURE

In the design it is assumed that

(a) entire section resists vertical loads and
(b) compression flange with channel resists the horizontal forces.

The following steps may be followed in the design:

1. With suitable positioning of crane, determine maximum moment and shear force on gantry girder. Add impact load contribution to it. Though the position for maximum moment due to wheel load is slightly away from the centre of the girder (under the wheel), it is just added to maximum moment due to *udl* on girder and design moment is found.

2. Calculate horizontal bending moment due to surge load.
3. Calculate shear forces due to vertical and horizontal forces.
4. Selection of trial section: The design is by the method of trials. A trial section is to be selected for which the following guidelines are useful:
   (a) The economic depth is about $\frac{1}{12}$ th span
   (b) Compression flange width may be kept $\frac{1}{25}$ th span
   (c) The moment capacity for vertical loads should be about 40% more than the moment due to vertical load, so that section can resist combined moment safely.
5. Calculate I_{zz}, I_{yy} and Z_p of the trial section selected.
6. Check for moment capacity of the section.
7. Check for buckling resistance as per clause 8.2.2 of IS Code.
8. Check for biaxial bending.
9. Check for shear capacity.
10. Check for web buckling and web bearing.
11. Check for deflection.
12. Design the welds.

The design procedure is illustrated with the example below.

Example 11.1

Design a simply supported gantry girder to carry an electric overhead travelling crane, given:

Span of gantry girder = 6.5 m

Span of crane girder = 16 m

Crane capacity = 250 kN

Self weight of crane girder excluding trolley = 200 kN

Self weight of trolley = 50 kN

Minimum hook approach = 1.0 m

Distance between wheels = 3.5 m

Self weight of rails = 0.3 kN/m

Solution:

1. *Moments*

Load for Maximum Moment:

Weight of trolley + lifted load = 250 + 50 = 300 kN

Self weight of crane girder = 200 kN.

For maximum reaction on gantry girder, the moving load should be as close to gantry as possible. Figure 11.6 shows the load position.

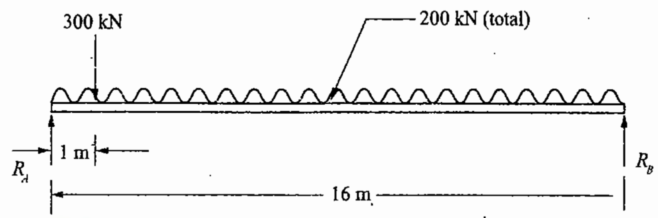

Figure 11.6

$$R_A = \frac{300 \times 15 + 200 \times 8}{16} = 381.25 \text{ kN}$$

This load is transferred to gantry girder through two wheels, the wheel base being 3.5 m.

$\therefore$ Load on gantry girder from each wheel $= \frac{381.25}{2} = 190.63$ kN

Factored wheel load = 190.63 × 1.5 = 286 kN.

Maximum moment due to moving loads occur under a wheel when the c.g. of wheel load and the wheel are equidistant from the centre of girder. This is shown in Fig. 11.7.

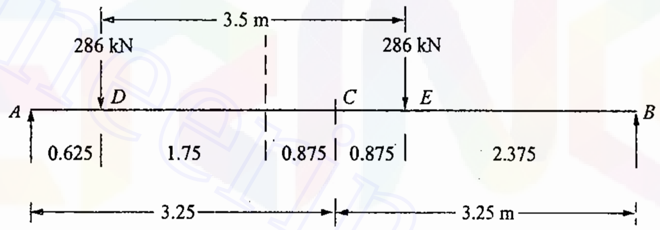

Figure 11.7

$$R_B = \frac{286 \times 0.625 + 286(3.25 + 0.875)}{6.5} = 209 \text{ kN}.$$

Max moment M_E = 209 × 2.375 = 496.375 kN-m.

Moment due to impact = 0.25 × 496.375 = 124.094 kN-m.

Assume self weight of girder = 2 kN/m.

$\therefore$ Dead load due to self weight + rails = 2 + 0.3 = 2.3 kN/m

$\therefore$ Factored DL = 2.3 × 1.5 = 3.45 kN/m.

Moment due to DL = $3.45 \times \frac{6.5^2}{8} = 18.22$ kN-m.

Factored moment due to vertical loads

$$M_z = 496.375 + 124.094 + 18.22 = 638.689 \text{ kN-m.}$$

Maximum moment due to horizontal force (surge):

Horizontal force transverse to rails = 10% of weight of trolley plus load lifted

$$= \frac{10}{100}(250 + 50) = 30 \text{ kN}.$$

Assuming double flamed wheels, this is distributed over 4 wheels

$\therefore$ Horizontal force on each wheel = 7.5 kN

Factored horizontal force on each wheel = 1.5 × 7.5

= 11.25 kN

For maximum moment in gantry girder the position of loads is same as shown in Fig. 11.7 except that it is horizontal. Hence by proportioning we get,

$$M_y = \frac{11.25}{286} \times 496.375 = 19.525 \text{ kN-m}$$

Shear Forces:

For maximum shear force on the girder, the trailing wheel should be just on the girder as shown in Fig. 11.8.

$\therefore$ Vertical shear due to wheel loads = $286 + \frac{286 \times 3.0}{6.5} = 418$ kN.

Vertical shear due to impact = 0.25 × 418

= 104.5 kN.

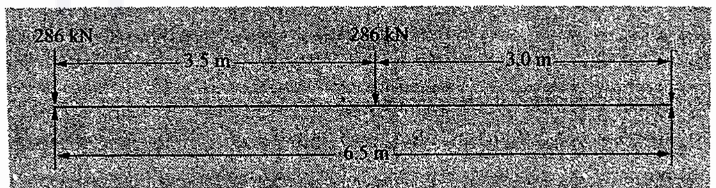

Figure 11.8

Vertical shear due to self weight = $3.45 \times \frac{6.5}{2} = 11.21$ kN

$\therefore$ Total vertical shear = 418 + 104.5 + 11.21 = 533.71 kN.

By proportioning lateral shear due to surge = $\frac{11.25}{286} \times 148.4 = 16.44$ kN.

Preliminary Section:

$$\frac{L}{12} = \frac{6500}{12} = 541.7 \text{ mm}$$

$$\frac{L}{25} = \frac{6500}{25} = 260 \text{ mm}$$

Let us try ISWB 600 with ISMC 300 on compression flange as shown in Fig. 11.9.

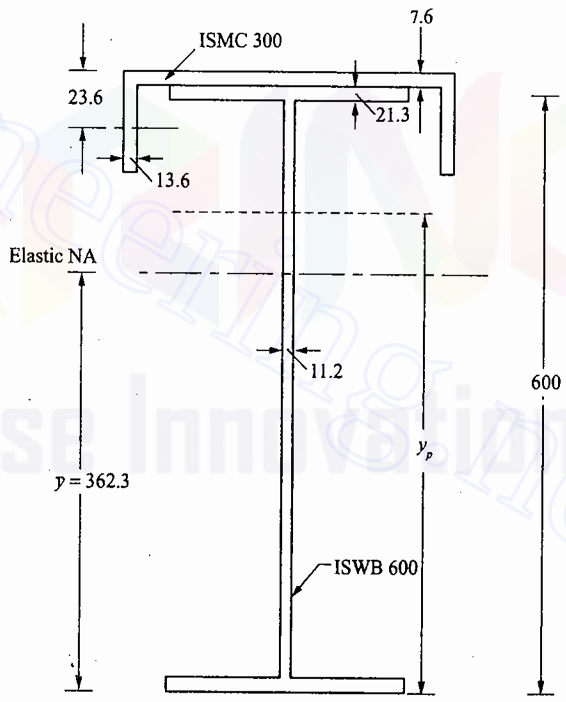

Figure 11.9

Properties of ISWB 600 @ 1.312 kN/m

$A = 17038 \text{ mm}^2$

$b = 250$ mm

$t_f = 21.3$ mm

$t_w = 11.2$ mm

$I_{zz} = 106198.5 \times 10^4 \text{ mm}^4$

$I_{yy} = 4702.5 \times 10^4 \text{ mm}^4$

Properties of ISMC 300

$A = 4564 \text{ mm}^2$

$b = 90$ mm

$t_f = 13.6$ mm

$t_w = 7.6$ mm

$I_{zz} = 6362.6 \times 10^4 \text{ mm}^4$

$I_{yy} = 310.8 \times 10^4 \text{ mm}^4$

$C_{yy} = 23.6$ mm

Let distance of *N-A* from the extreme fibre of tension flange be $\bar{y}$.

Then,

$$\bar{y} = \frac{17038 \times 300 + 4564 \times (600 + 7.6 - 23.6)}{17038 + 4564} = 360.0 \text{ mm}$$

$$I_{zz} = 106198.5 \times 10^4 + 17038\,(360 - 300)^2 + 310.8 \times 10^4 + 4564 \times (584 - 360)^2$$

$$= 1127.452 \times 10^6 \text{ mm}^4$$

$$Z_e = \frac{I_{zz}}{y_{max}} = \frac{1127.452 \times 10^6}{360.0} = 313.18 \times 10^4 \text{ mm}^4$$

For compression flange about *y-y* axis,

$$I = \frac{1}{12} \times 21.3 \times 250^3 + 6362.6 \times 10^4 = 9136.04 \times 10^4 \text{ mm}^4.$$

$$Z_{ey} \text{ for compression flange} = \frac{9136.04 \times 10^4}{150} = 609.069 \times 10^3 \text{ mm}^3$$

Plastic Modulus of Section: [Ref. Fig. 11.10]

Total area of the section $= 17038 + 4564 = 21602 \text{ mm}^2$

Let plastic *N-A* be at a distance Y_p from tension flange. Then

$$(Y_p - 21.3) \times 11.2 + 250 \times 21.3 = \frac{A}{2} = \frac{21602}{2}$$

$$\therefore \quad Y_p = 510.2 \text{ mm.}$$

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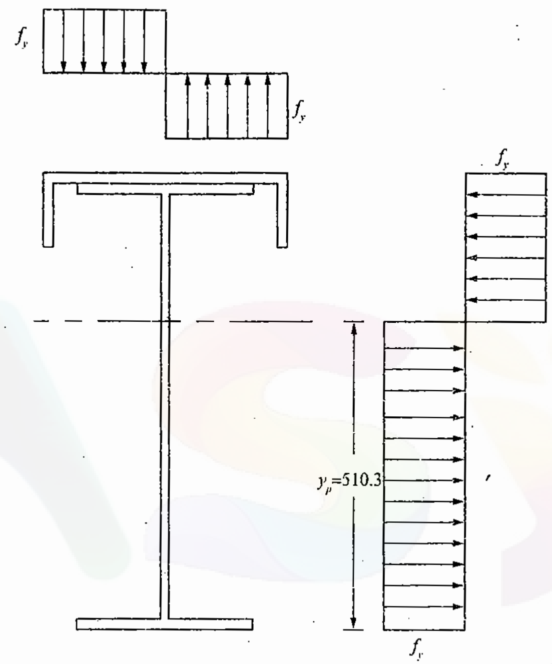

Figure 11.10

$$\therefore \quad M_p = \Sigma \text{ Moment of forces at yield about plastic } N\text{-}A$$

$$= 21.3 \times 250 \left(510.2 - \frac{21.3}{2}\right) f_y + \frac{(510.2 - 21.3)^2}{2} \times 11.2\, f_y$$

$$+ \frac{(600 - 21.3 - 510.2)^2}{2} \times 11.2\, f_y + 21.3 \times 250 \left(600 - \frac{21.3}{2} - 510.2\right) f_y$$

$$+ 4564\,(600 + 13.6 - 23.6 - 510.2)\, f_y$$

$$= 4686450\, f_y$$

$$\therefore \quad Z_p = \frac{M_p}{f_y} = 4686450 \text{ mm}^3.$$

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For top flange,

$$Z_{py} = \frac{M_p}{f_y} = \frac{1}{4}\times 21.3\times 250^2 + \frac{1}{4}(300-2\times 13.6)^2\times 7.6 + 2\times 90\times 13.6\left(150-\frac{13.6}{2}\right)$$

$$= 824.764\times 10^3 \text{ mm}^3.$$

Check for Moment Capacity:

$$\frac{b}{t} \text{ of flange of ISWB 600} = \frac{250-11.2}{2\times 21.3} = 5.6 < 8.4$$

$$\frac{d}{t} \text{ of web of ISWB 600} = \frac{600-2\times 21.3}{11.2} = 49.76 < 84$$

$$\text{and } \frac{b}{t} \text{ of flange of channel} = \frac{90-7.6}{13.6} = 6.06 < 8.4$$

Hence it is a plastic section.

Local moment capacity for bending in vertical plane:

$$M_{dz} = \frac{f_y z_p}{1.1} = \frac{250}{1.1}\times 4686450 = 1065.1\times 10^6 \text{ N-mm}$$

$$= 1065.1 \text{ kN-m.}$$

$$\frac{1.2 z_e f_y}{1.1} = \frac{1.2\times 313.18\times 10^4\times 250}{1.1} = 854.127\times 10^6 \text{ N-mm}$$

$$= 854.127 \text{ kN-m.}$$

$\therefore$ $M_{dz} = 854.127$ kN-m.

For top flange: $$M_{dz} = \frac{f_y z_p}{1.1} = \frac{250}{1.1}\times 824.764\times 10^3 = 187.446\times 10^6 \text{ N-mm}$$

$$= 187.446 \text{ kN-m}$$

$$\frac{1.2 z_e f_y}{1.1} = \frac{1.2\times 609.069\times 10^3\times 250}{1.1} = 166.11\times 10^6 \text{ N-mm}$$

$$= 166.11 \text{ kN-m}$$

$\therefore$ For top flange, $M_{dz} = 166.11$ kN-m.

Check for Combined Local Capacity:

$$\frac{M_z}{M_{dz}} + \frac{M_y}{m_{dy}} \le 1$$

$$\frac{638.689}{854.127} + \frac{19.525}{166.11} = 0.865 < 1$$

Hence adequate.

Check for Buckling Resistance [clause 8.2.2.1]:

$M_d = \beta_b Z_p f_{bd}$

For plastic section $\beta_b = 1.0$

$$\therefore \quad M_d = Z_p f_{bd}$$

$$f_{cr} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1+\frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

$L_{LT} = 6500$ mm $\quad E = 2\times 10^5$ N/mm^2 $\quad h_f = 600 + 7.6 = 607.6$ mm

$I_y = 4702.5\times 10^4 + 6362.6\times 10^4 = 11065.1\times 10^4$ mm^4

$A = 17038 + 4564 = 21602$ mm^2.

$$\therefore \quad r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{11065.1\times 10^4}{21602}} = 71.57 \text{ mm}$$

$$\therefore \quad f_{crb} = \frac{1.1\times \pi^2\times 2\times 10^5}{(6500/71.57)^2}\left[1+\frac{1}{20}\left(\frac{6500/71.56}{607.6/21.3}\right)^2\right]^{0.5}$$

$$= 323.06 \text{ N/mm}^2.$$

(*Note:* Table 14 of IS 800 also may be used to find f_{crb}).

From Table 13(a),

$$f_{bd} = 167.8 \text{ N/mm}^2$$

$\therefore$ $M_{dz} = 1.0\times 167.8\times 4686450 = 786.39\times 10^6$ N-mm

$= 786.39$ kN-m > 638.689 kN-m

Hence the section is adequate.

Check for Biaxial Bending:

$$M_{dy} = \frac{f_y Z_y}{1.1}$$

$$Z_y = \frac{I_y}{150} = \frac{11065.1\times 10^4}{150} = 737.67\times 10^3 \text{ mm}^3.$$

$$\therefore \quad M_{dy} = \frac{250}{1.1} \times 737.67 \times 10^3 = 167.65 \times 10^6 \text{ N-mm}$$

$$= 167.65 \text{ kN-m}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{638.689}{786.54} + \frac{19.525}{167.65}$$

$$= 0.928 < 1$$

Hence adequate.

Check for Shear:

$$V_z = 533.71 \text{ kN.}$$

$$\text{Shear capacity} = \frac{A_v f_{yw}}{\sqrt{3} \times 1.1} = \frac{600 \times 11.6 \times 250}{\sqrt{3} \times 1.1}$$

$= 913 \times 10^3$ N $= 913$ kN > 533.71 kN **O.K.**

$0.6 \times 913 = 547.8$, slightly less than V. Considering it as high shear case may be ignored.

Hence there is no reduction in moment capacity. Therefore moment capacity is adequate as found earlier.

Weld Design:

$$\text{Shear stress} = q = \frac{V}{bI}(a\bar{y})$$

$$\therefore \quad \text{Shear per unit length} = \frac{V}{I}(a\bar{y})$$

$V = 549.9$ kN

$a =$ Area of channel $= 4564 \text{ mm}^2$

$I = I_z = 1207.28 \times 10^6 \text{ mm}^4$

$\bar{y}$ = Distance of L.G. of channel from N-$A = 600 + 7.6 - 23.6 - 362.3 = 224.3$ mm

$$\therefore \quad \text{Shear force per unit length } q = \frac{533.71 \times 10^3}{1127.452 \times 10^6}(4564 \times 224.3)$$

$$= 484.6 \text{ N/mm}$$

If 's' is the size of weld provided on each side, then shear strength of weld

$$= 2s \times 0.7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 265.12\, s \text{ N/mm}$$

Equating it to shear force, we get

$$265.12\, s = 484.6$$

$$s = 1.83 \text{ N/mm}$$

Hence provide 5 mm intermittent fillet weld (which is minimum) on both sides, % welding $\frac{1.83}{5} \times 100 = 36.6$. Provide 5 mm, 40%, weld.

Check for Web Buckling:

$d = 600 - 2(21.3 + 17) = 523.4$ mm, $t = 11.2$ mm

$\therefore\ d/t = \frac{523.4}{11.2} = 46.7 < 67$. Hence no need to check (clause 8.4.2.1)

Check for Deflection:

At working load, deflection is to be limited to $\frac{L}{750}$. For maximum deflection wheel load is as shown in Figures 11.1(a) Fig. 11.11(b) shows the conjugate beam with $\frac{M}{EI}$ diagram.

Reaction in conjugate beam

$$= \frac{1}{2} \text{ total } \frac{M}{EI} \text{ diagram}$$

$$= \frac{1}{2} \times 1.5 \times \frac{286}{EI} + \frac{286}{EI} \times 1.75 = \frac{715}{EI}$$

Maximum deflection occurs at mid span = Moment of $\frac{M}{EI}$ load in conjugate beam

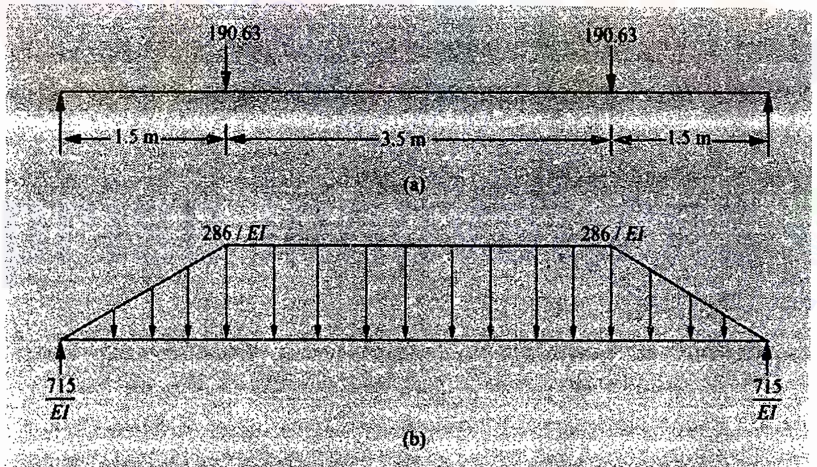

Figure 11.11

$$EI\Delta = 715\times\frac{6.5}{2} - \frac{1}{2}\times 286\times 1.5\times 2.75 - 1.75\times 286\times\frac{1.75}{2}$$
$$= 1295.9$$

Taking EI in kN-m^2 unit,

$$EI = 2\times10^5\times1207.8\times10^6\times\frac{1}{10^9} = 200\times1207.8 \text{ kN-m}^2.$$

$$\therefore \quad \Delta = \frac{1295.9}{200\times1127.452} = 5.75\times10^{-3} \text{ m} = 5.75 \text{ mm}$$

Permissible $\Delta = \frac{L}{750} = \frac{6500}{750} = 8.66$ mm.

$\therefore$ Deflection requirement is satisfied.

Hence use ISWB 600 with ISMC 300 on compression flange as shown in Fig. 11.9.

* * *

Questions

1. The following data refers to a gantry girder on which an electrically operated crane of capacity 200 kN moves

Span of gantry girder	= 6.0 m
Span of crane girder	= 18 m
Crane capacity	= 200 kN
Self weight of crane girder	= 180 kN
Self weight of trolley	= 75 kN
Minimum hook approach	= 1.0 m
Distance between wheels	= 3.5 m
Self weight of rails	= 0.3 kN/m

Determine:

(1) the maximum moment and shear forces due to vertical and horizontal loads
(2) check whether ISMB 600 with ISMC 300 on compression flange is adequate to

(a) carry moment
(b) carry shear force
(c) in buckling resistance
(d) in limiting deflections.

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12

DESIGN OF ROOF TRUSSES

Large column free areas are required for auditoriums, assembly halls, workshops etc. To get such column free area one of the commonly used roofing system is to provide a set of steel roof trusses, interconnected with purlins which in turn support GI (Galvanised Iron) or A.C. (Asbestos Cement) sheets. The roof trusses are supported on walls or a series of columns. Figure 12.1 shows a typical roof truss covering system.

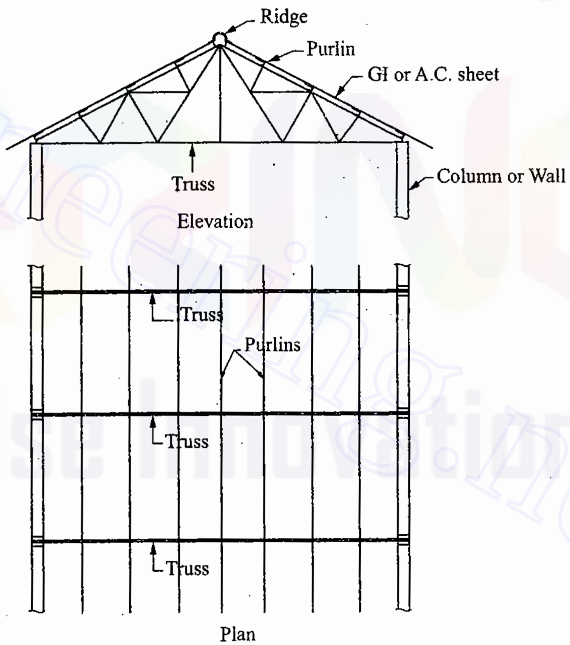

Figure 12.1

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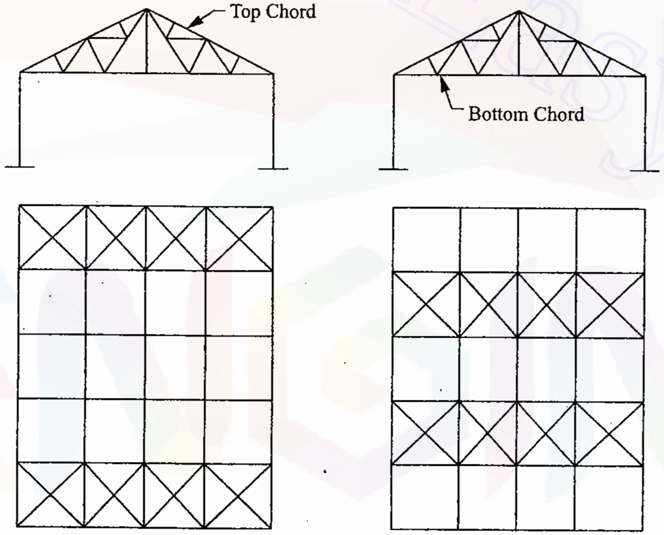
12.1 BRACINGS

The trusses are sufficiently strong to transmit the forces acting in their plane. However they offer little resistance to forces acting at right angles to their plane. Such force is mainly due to wind blowing parallel to ridges. The resistance offered by purlins to such forces may not be sufficient. Hence bracings are provided at two levels i.e. at the level of top chord and at the level of bottom chord. Figure 12.2(a) shows bracings at top chord level, which is in the end pair by using angle iron ISA $90 \times 60 \times 8$ mm. Figure 12.2(b) shows bracing at bottom chord level. They consist of longitudinal strip with $\frac{L}{r} \nless 250$ connecting at panel points. Diagonal bracings are also provided in the last but one panel from both ends. For very long buildings additional diagonal bracings are provided at every 4 to 5 bays.



(a) Plan at top chord level (b) Plan at bottom chord level

Figure 12.2 Bracing system.

12.2 TYPES OF ROOF TRUSSES

Triangle is the primary pin jointed frame which has stable figure. Hence all trusses should comprise triangular figures. Various types of trusses are used. Figure 12.3 shows different types of trusses and also their suitability for different spans.

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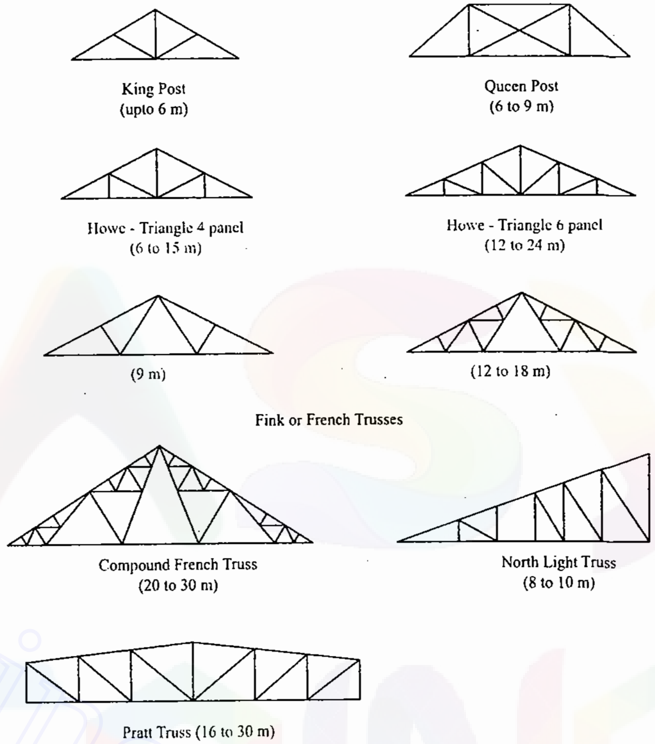

Figure 12.3 Types of trusses.

12.3 NOMENCLATURE OF MEMBERS OF TRUSSES

The nomenclatures used for the various members of a truss are explained with respect to the Fig. 12.4 below:

1. *Top Chord Members*: These are the members along the uppermost line of truss starting from one end of support to another end and passing through the peak. These members are also known as rafters. They directly support purlins. They are mainly subjected to axial compression due to live load and dead load. If the purlins are not supported at joints these members are subjected to bending stresses also. Members 1-2, 2-3, 3-4, 4-5, 5-6, 6-7, 7-8 and 8-9 belong to this category.
2. *Bottom Chord Members*: These are the members extending from one end of the support to another end along the lowermost line of truss. These members are also known as main ties. They are mainly subjected to tensile forces due to dead load and live loads. Members 1-10, 10-11, 11-12, 12-13, 13-14 and 14-9 belong to this group. The bottom chord may be horizontal or may have some camber.

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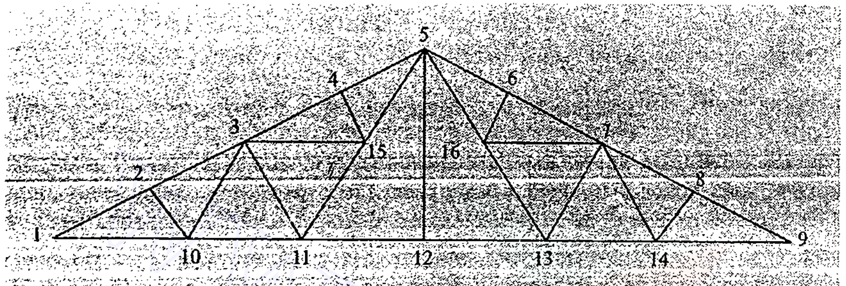

Figure 12.4

3. *Struts*: The members which do not belong to top or bottom chord and subjected to compressive forces are called struts. Depending upon the relative magnitude of forces they take, they are further classified as main struts and minor struts. Members 3-11 and 7-13 may be called main struts while the members 2-10, 4-15, 6-16 and 8-14 may be called minor struts.
4. *Slings*: The members which do not belong to top/bottom chord but are mainly subjected to tension are designated as slings. Depending upon the relative magnitude of tensile forces they carry, slings are further classified as main slings and minor slings. The members 5-15, 6-16, 15-11 and 16-13 belong to the group of main slings while the members 3-10, 3-15, 7-14 and 7-16 are minor slings.
5. *Sag Tie*: If no load is acting at joint 12, the members 5-12 is not subjected to any force. Usually this is the loading case in the truss. Even then this member is provided to reduce the sag in the member 11-13.

12.4 PITCH OF TRUSSES

It is defined as the ratio of height of the truss to the span. A minimum pitch of $\frac{1}{6}$ is to be maintained for GI sheet covering and $\frac{1}{12}$ is to be maintained for A.C. sheet covering. The preferable pitches are $\frac{1}{4}$ if snow load is expected and $\frac{1}{6}$ if snow load is not expected.

12.5 SPACING OF TRUSSES

The distance between the two consecutive trusses is called spacing of truss. The spacing of trusses is governed by the size of space to be covered by roof. As the spacing increases, the number of trusses may reduce but the cost of purlins increase. Wherever possible, the following guidelines may be used in deciding the spacing of trusses:

(a) 3 to 4.5 m upto 15 m span.
(b) 4.5 m to 6.0 m for 15-30 m span.
(c) For spans more than 40 m, spacing of 12 to 15 m may be used with cross trusses replacing purlins.

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12.6 PURLINS

As far as possible purlins should be located on panel points of top chord members. However it depends upon the type of roofing materials also. Generally the spacing of purlins varies from 1.35 m to 2.0 m.

Angle iron purlins are used for smaller spacing of trusses (3 to 4 m). For medium spacing (4 to 5 m) one can use channels and for still larger spans, I-sections may be used. If angles are used, outstanding legs are at top and lug angles are used to connect the purlins to rafters.

12.7 SHEETINGS

Commonly used sheetings are GI and A.C.

12.7.1 GI Sheets

Corrugated iron sheets are galvanized for protection against corrosion and are used as roof coverings. The common sizes of GI sheets are:

(i) 8 corrugations, 75 mm wide and 19 mm deep which have overall width of 660 mm.
(ii) 10 corrugations, 75 mm wide and 19 mm deep, which have overall width of 810 mm.

The sheets are available in the gauges 16, 18, 20, 22 and 25 [*Note:* thickness = 25/gauge mm]

The sheets are available in lengths 1.8 m, 2.2 m, 2.5 m, 2.8 m and 3.0 m. The sheets should be used with the following overlaps:

Side laps: 1, $1\frac{1}{2}$ or 2 corrugations
End laps: 100 mm, if slope is more and 150 mm, if slope is less than 20°.

For lesser overlaps suitable sealing should be made. The sheets should be fastened to purlins or sheeting rails by 8 mm diameter hook bolts at a maximum pitch of 350 mm.

The spacing of purlins depends upon the applied loading, thickness of sheets and length of sheets. For common loading, the thicknesses of sheetings are so fixed that, with required overlaps the sheetings can be used fully.

12.7.2 A.C. Sheets

Asbestos cement sheets are better insulators for sun's heat compared to GI sheet. They are used commonly in the factories and godowns. They are available in two common shapes viz. corrugated and traffold. They are available in the lengths of 1.75, 2.0, 2.5 and 3.0 m. They are available in thicknesses of 6 mm and 7 mm. The maximum permissible spacing is 1.4 m for 6 mm sheets and 1.6 m for 7 mm sheets. They are to be used with a longitudinal overlap of 150 mm and a side overlap of one corrugation spacing of purlins are to be adjusted such that as far as possible the cutting of sheets is avoided.

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12.8 LOADS

The main loads on trusses are

(i) Dead Loads
(ii) Imposed Loads
(iii) Wind Loads
(iv) Other Loads.

12.8.1 Dead Loads

It includes the weight of sheetings, purlins, bracings, self weight and other loads suspended from trusses.

The unit weight of various materials are given in IS 875 – part I. The following values may be noted:

GI sheets: 85 N/m^2
A.C. sheets: 130 N/m^2

In general the roof covering weight including laps, connector etc. may be taken as

(i) 100-150 N/m^2 for GI sheeting
(ii) 170-200 N/m^2 for A.C. sheeting.

Weight of purlins should be added after designing purlin. Generally it works out 100-120 N/m^2 of plan area.

On lower panel points additional occasional load may be considered. Such load is due to electrical fixtures, fans etc. This may be assumed to be 5 to 10 kN, distributed over lower panel points. If the false ceiling is to be suspended that load should be estimated separately.

There are various formulae suggested in literature to assume self weight of truss. However, since it is a small percentage of total load any one of them may be used. The following are the two formulae commonly used for a truss of span 'L'.

(i) $w = 20 + 6.6\,L$ N/m^2 for a LL of 2 kN/m^2

If live load is more, the above value is to be increased by $LL/2.0$.

(ii) $w = 10\left(\frac{L}{3}+5\right)\frac{s}{4}$ N/m^2

where s = spacing of trusses.

12.8.2 Imposed Load (Live Load)

Normally no access is provided for sloping roofs with sheets. In such cases IS 875 part II makes the following provisions for live loads for the design of sheets and purlins.

Upto 10° slope: 0.75 kN/m^2

For more than 10° slope: 0.75 – 0.02 (θ – 10), where θ is slope of sheeting.

However a minimum of 0.4 kN/m^2 live load should be considered in any case.

For the design of trusses the above live load may be reduced to $\frac{2}{3}$ rds.

The purlins and sheets should be checked to support a concentrated load of 0.9 kN at the worst position.

12.8.3 Wind Load

IS 875 part 3 gives guidelines to determine wind forces on different components of buildings. It consists of the following steps:

(a) Determine basic wind speed.
(b) Obtain design wind speed.
(c) Calculate design wind pressure.
(d) Calculate wind pressure on roof.

These steps are explained below:

(a) Basic Wind Speed (V_b)

For finding basic wind pressure in any place in India, IS 875 (part 3) divides the country into six zones as shown in Fig. 12.5. It is based on peak gust velocity averaged over a short time interval of about 3 seconds studied over a period of 50 years. The values correspond to the speed at 10 m height above ground level and in an open terrain. For important cities basic wind pressure are given in the code. It may be observed that highest basic wind speed is 55 m/s and the lowest is 33 m/s.

(b) Design Wind Speed (V_z)

The design wind speed for any site may be obtained as:

$$V_z = k_1\, k_2\, k_3\, V_b$$

where k_1 = risk coefficient

k_2 = terrain, height and structure size factor

k_3 = topography factor

(i) *Risk Coefficient* (k_1)

Depending upon the importance of the building and basic wind speed IS 875 has developed an equation to determine risk coefficient k_1 for different types of buildings. Finally it gives the values in the tabular form as shown in Table 12.1.

(ii) *Terrain, Height and Structure Size factor* (k_2)

This coefficient depends upon the terrain of the building site, height of building and the class of building.

Terrain, in which a specific structure stands shall be assessed as belonging to one of the following categories:

Category 1: Exposed open terrain with few or no obstructions and in which average height of any objects surrounding the structure is less than 1.5 m.

Figure 12.5 Basic wind speed in m/s (based on 50-year return period) as per IS 875 (Part 3).

Category 2: Open terrain with well scattered obstructions having heights generally between 1.5 and 10 m.

Category 3: Terrain with numerous closely spaced obstructions having the size of buildings upto 10 m in height with or without a few isolated tall structures.

Category 4: Terrain with numerous large high closely spaced obstructions.

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Table 12.1 Risk coefficient for different classes of structures in different wind speed zones (*Clause 4.4.3.1*) (Table 4 of IS 875)

Class of Structure	Mean Probable Design Life of Structure in Years	k_1 Factor for Basic Wind Speed (m/s) of					
		33	39	44	47	50	55
All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
Temporary sheds, structure such as those used during construction operations (for example, form-work and falsework), structure during construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
Buildings and structure presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded area, farm buildings, other than residential buildings	25	0.94	0.92	0.91	0.90	0.90	0.89
Important buildings and structures, such as hospitals, communications buildings towers and power plant stuctures	100	1.05	1.06	1.07	1.07	1.08	1.08

To incorporate structure size factors the buildings are classified into the following three different classes:

Class A: Buildings having maximum dimension (greatest horizontal or vertical) less than 20 m.
Class B: Buildings having maximum dimension between 20 m and 50 m.
Class C: Buildings having maximum dimension greater than 50 m.

The factor k_2 depends upon the height of the building also. IS 875 (part 3) gives the following table (Table 12.2) to determine the coefficient k_2.

(iii) *Topography Factor* (k_3)

This factor is to account for the topographic features which influence design wind speed. It accounts for the topographic features, such as hills, valleys, cliffs, ridges etc.

Extent of Topographic Feature: IS 875 – suggests the extent of topographic feature as below (Fig. 12.6)

Let L = actual length of the upward wind slope.

z = effective height of the feature.

θ = upwind slope in the wind direction.

Then the influence of the topographic factor should be considered over an extent 1.5 *Le* to 2.5 *Le* towards upwind and downwind respectively as shown in the figure, where *Le* is the effective horizontal length of the hill which depends upon the slope as indicated below:

Table 12.2 k_2 factors to obtain design wind speed variation with height in different terrains for different classes of buildings structures [*Clause 4.4.3.2 (b)*] (Table 5 in IS 875)

Height (m)	Terrain Category 1 Class			Terrain Category 2 Class			Terrain Category 3 Class			Terrain Category 4 Class		
	A	B	C	A	B	C	A	B	C	A	B	C
10	1.05	1.03	0.99	1.00	0.98	0.93	0.91	0.88	0.82	0.80	0.76	0.67
15	1.09	1.07	1.03	1.05	1.02	1.97	0.97	0.94	0.87	0.80	0.76	0.67
20	1.12	1.10	1.06	1.07	1.05	1.00	1.01	0.98	0.91	0.80	0.76	0.67
30	1.15	1.13	1.09	1.12	1.10	1.04	1.06	1.03	0.96	0.97	0.93	0.83
50	1.20	1.18	1.14	1.17	1.15	1.10	1.12	1.09	1.02	1.10	1.05	0.95
100	1.26	1.24	1.20	1.24	1.22	1.17	1.20	1.17	1.10	1.20	1.15	1.05
150	1.30	1.28	1.24	1.28	1.25	1.21	1.24	1.21	1.15	1.24	1.20	1.10
200	1.32	1.30	1.26	1.30	1.28	1.24	1.27	1.24	1.18	1.27	1.22	1.13
250	1.34	1.32	1.28	1.32	1.31	1.26	1.29	1.26	1.20	1.28	1.24	1.16
300	1.35	1.34	1.30	1.34	1.32	1.28	1.31	1.28	1.22	1.30	1.26	1.17
350	1.37	1.35	1.31	1.36	1.34	1.29	1.32	1.30	1.24	1.31	1.27	1.19
400	1.38	1.36	1.32	1.37	1.35	1.30	1.34	1.31	1.25	1.32	1.28	1.20
450	1.39	1.37	1.33	1.38	1.36	1.31	1.35	1.32	1.26	1.33	1.29	1.21
500	1.40	1.38	1.34	1.39	1.37	1.32	1.36	1.33	1.28	1.34	1.30	1.22

Note – Intermediate values may be obtained by linear interpolation, if desired. It is permissible to assume constant wind speed between two heights for simplicity.

Slope	*Le*
$3° < \theta \le 17°$	L
$\theta > 17°$	$\dfrac{z}{0.3}$

Topography Factor: The topographic factor k_3 is given by the following:

$$k_3 = 1 + Cs$$

where C has the following values:

Slope	C
$3° < \theta \le 17°$	$1.2\left(\dfrac{z}{L}\right)$
$\theta > 17°$	0.36

s is a factor to be determined from Fig. 12.7 for cliff and escarpments and from Fig. 12.8 for hills and ridges. In these figures 'X' is the distance of the site from crest of hill.

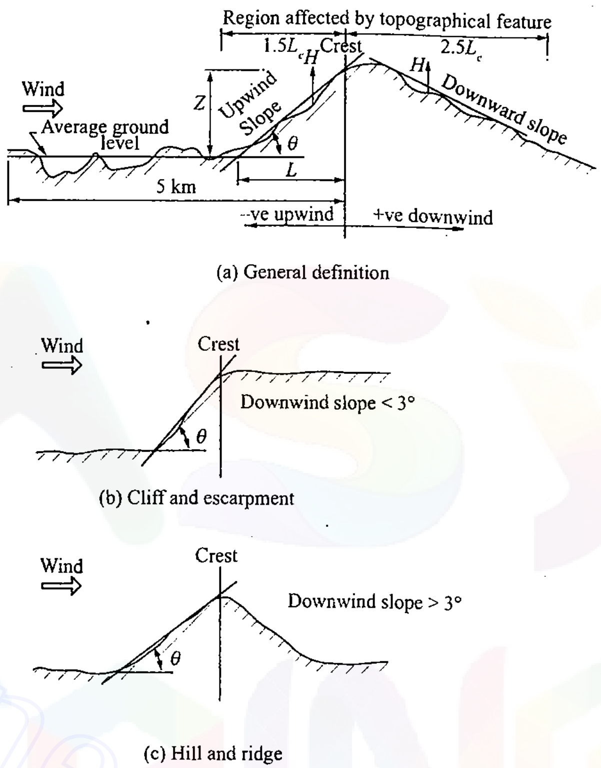

Figure 12.6 Topographic features.

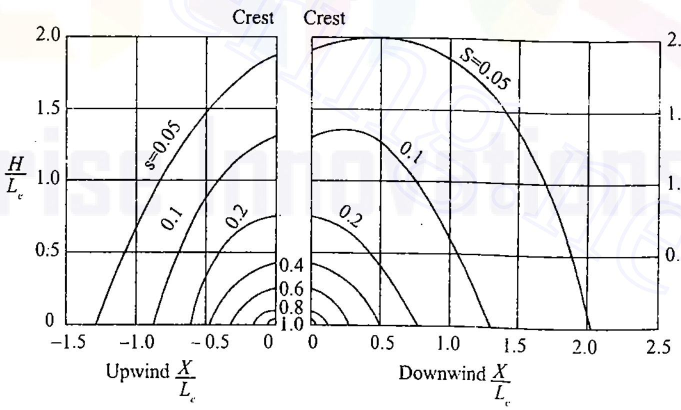

Figure 12.7 Factors for cliff and escarpment.

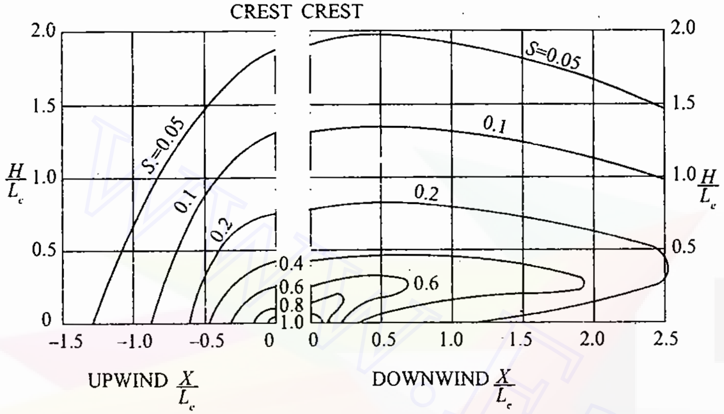

Figure 12.8 Factors for ridge and hill.

(c) *Wind Pressure*

The design wind pressure at any height above mean ground level shall be calculated using the following expression

$$P_z = 0.6\, V_z^2 \text{ N/m}^2$$

where,

P_z = design wind pressure in N/m^2 at height z and

V_z = design wind velocity in m/s at height z.

(d) *Wind Pressure on Roof*

For calculating the wind load on individual structural elements it is essential to take into account of pressure difference between opposite faces of such elements. If internal as well as external pressures are found, then wind load acting in a direction normal to the individual structural element or cladding unit is:

$$F = (C_{pe} - C_{pi})\, A p_d$$

where,

C_{pe} = external pressure coefficient

C_{pi} = internal pressure coefficient

A = surface area of structural element or cladding unit in m^2 and

p_d = design wind pressure in N/m^2.

Positive wind coefficient indicates the force is towards the structural element and negative coefficient indicates it is away from the structural element. Hence the +ve senses of internal and external wind pressures are as shown in Fig. 12.9.

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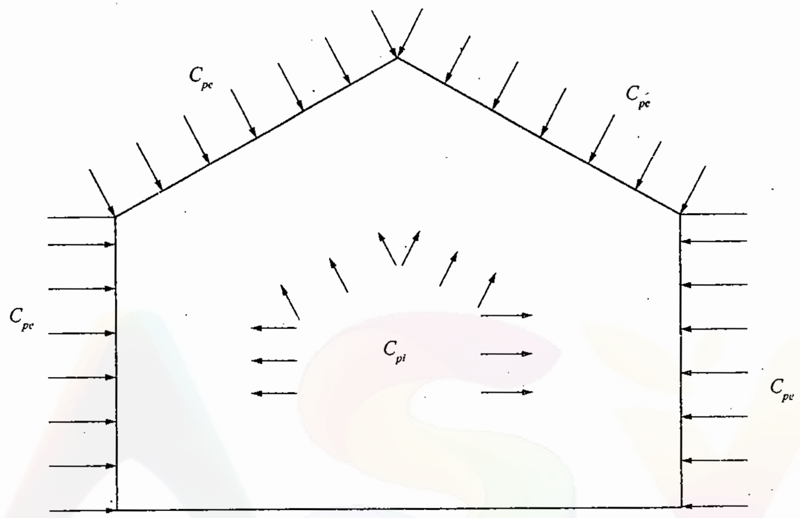

Figure 12.9 Positive senses of C_{pi} and C_{pe}.

The external wind pressure coefficient depends upon

(a) the shape of the roof
(b) size of the building
(c) slope of the roof
(d) wind angle and
(e) the zone of the structural element in the building.

IS 875 gives specifications for pitched roofs of rectangular clad buildings, monoslope roofs of rectangular clad buildings, canopy roofs, curved roofs and saw tooth roofs of multispan buildings. For pitched roof of rectangular clad building it gives the values for eight zones-four main zones four local zones. These values are shown in Table 12.3.

Internal air pressure in a building depends upon the degree of permeability of the air flow. For both internal pressure of +ve as well as –ve values the structural element should be safe. Table 12.4 shows the internal pressure coefficients for different types of the building.

IS 875 gives the values of C_{pi} for buildings with openings on one side also.

12.8.4 Other Loads

Other possible loads on the structure also should be considered in the design. One such load is snow load. In India, such loads are expected in some hilly regions. This type of loading has been already discussed in chapter 1. For details one can refer to IS 875 (part 3).

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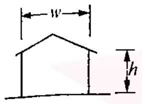
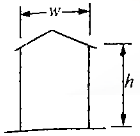
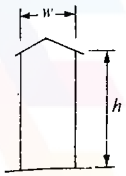
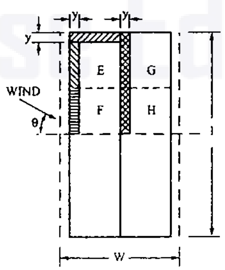
Table 12.3 External pressure coefficients (C_{pe}) for pitched roofs of rectangular clad buildings (Table 8 in the code)

Building Height Ratio	Roof Angle	Wind Angle θ 0°		Wind Angle θ 90°		Local Coefficients			
		EF	GH	EG	FH				
	degrees								
$1/2 < h/w$ 	0	– 0.8	– 0.4	– 0.8	– 0.4	– 2.0	– 2.0	– 2.0	–
	5	0.9	– 0.4	– 0.8	0.4	1.4	– 1.2	– 1.2	– 1.0
	10	– 1.2	– 0.4	– 0.8	– 0.6	– 1.4	– 1.4		– 1.2
	20	– 0.4	– 0.4	– 0.7	– 0.6	– 1.0			– 1.2
	30	0	– 0.4	– 0.7	– 0.6	– 0.8			– 1.1
	45	+ 0.3	– 0.5	– 0.7	– 0.6				– 1.1
	60	+ 0.7	– 0.6	– 0.7	– 0.6				– 1.1
$1/2 < h/w$ 3/2 	0	– 0.8	– 0.6	– 1.0	– 0.6	– 2.0	– 2.0	– 2.0	–
	5	– 0.9	– 0.6	– 0.9	– 0.6	– 2.0	– 2.0	– 1.5	– 1.0
	10	– 1.1	– 0.6	– 0.8	– 0.6	– 2.0	– 2.0	– 1.5	– 1.2
	20	– 0.7	– 0.5	– 0.8	0.6	– 1.5	– 1.5	– 1.5	– 1.0
	30	– 0.2	– 0.5	– 0.8	– 0.8	– 1.0			– 1.0
	45	+ 0.2	– 0.5	– 0.8	– 0.8				
	60	+ 0.6	– 0.5	– 0.8	– 0.8				
$3/2 < h/w$ 6 	0	– 0.7	– 0.6	– 0.9	– 0.7	– 2.0	– 2.0	– 2.0	–
	5	– 0.7	– 0.6	– 0.8	– 0.8	– 2.0	– 2.0	– 1.5	– 1.0
	10	– 0.7	– 0.6	– 0.8	– 0.8	– 2.0	– 2.0	– 1.5	– 1.2
	20	– 0.8	– 0.6	– 0.8	– 0.8	– 1.5	– 1.5	– 1.5	– 1.2
	30	– 1.0	– 0.5	– 0.8	– 0.7	– 1.5			
	40	– 0.2	– 0.5	– 0.8	– 0.7	– 1.0			
	50	+ 0.2	– 0.5	– 0.8	– 0.7				
	60	+ 0.5	– 0.5	– 0.8	– 0.7				

Note 1 – h is the height to eaves or parapet, w is the lesser horizontal dimension of a building.

Note 2 Where no local coefficients are given the overall coefficients apply.



KEY PLAN

$y = h$ or $0.15\ w$, whichever is the lesser

Table 12.4 Internal pressure coefficient *Cpi*

Buildings with	Cpi
Low permeability (less than 5% openings in wall area)	± 0.2
Medium permeability (5–20 % opening in wall)	± 0.5
Large openings (openings > 20 % in wall)	± 0.7

Earthquake loads do not influence the design of pitched roofs with GI or A.C. sheets, since these are light weight roofs.

12.9 LOAD COMBINATIONS

The following load combinations are to be considered in the design of cladding and trusses:

1. Dead Load + Imposed Loads
2. Dead Load + Snow Load (if this is expected)
3. Dead Load + Wind Load (wind direction with $\theta = 0$ or $\theta = 90°$).

Example 12.1

A roof truss shed is to be built in Lucknow for an industry. The size of shed is 24 m × 40 m. The height of building is 12 m at the eves. Determine the basic wind pressure.

Solution:

From wind zone map of country (IS 875 part 3) the basic wind speed in Lucknow is

$$V_b = 47 \text{ m/sec.}$$

Risk Coefficient k_1: From Table 12.1, for all general buildings with probable design life of structure 50 years,

$$k_1 = 1.0$$

Terrain, Height and Structure Size Factor k_2:

Since the shed is in an industrial area, it may be considered belonging to *category* 3. Its greatest dimension being 40 m, it belongs to *class B structure*. For category 3, class B building

$$k_2 = 0.88 \quad \text{if} \quad h = 10 \text{ m.}$$

$$= 0.94 \quad \text{if} \quad h = 15 \text{ m.}$$

∴ For $h = 12$ m,

$$k_2 = 0.88 + (0.94 - 0.88)\frac{2}{5} = 0.904.$$

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Topography Factor k_3: In Lucknow, the ground near shed may be assumed plain.

$$\therefore \quad k_3 = 1 + Cs$$

where $c = \frac{z}{L} = 0$

$$\therefore \quad K_3 = 1.0$$

Design wind speed

$$V_z = K_1 K_2 K_3 V_b$$
$$= 1.0 \times 0.904 \times 1.0 \times 47$$
$$= 42.488 \text{ m/sec.}$$

Hence basic wind pressure

$$p_z = 0.6V_z^2 = 0.6 \times 42.488^2$$
$$= 1083 \text{ N/m}^2$$
$$p_z = 1.083 \text{ kN/m}^2 \qquad \textbf{Answer}$$

Example 12.2

A power plant structure having maximum dimension more than 60 m is proposed to be built on down hill side near Dehradun. The height of the hill is 400 m with a slope of 1 in 3. If the location is 250 m from the crest of the hill on downward slope, and its eve board is at a height of 9 m, determine the design wind pressure.

Solution:

From Fig. 12.5, the *basic wind pressure* at Dehradun = 39 m/sec.

Risk Coefficient (k_1): From Table 12.1, for power plant structure mean probable life of the structure is to be taken as 100 years and hence, $k_1 = 1.06$

Terrain, Height and Structure Size Factor (k_2):

Since height of building is 9 m, it belongs to category 2 structure.

The size of building is more than 60 m. Hence it belongs to class '*C*' structure.

$\therefore$ From Table 12.2, we find

$$k_2 = 0.93$$

Topography Factor (k_3):

The slope is 1 in 3.

$$\therefore \quad \tan\theta = \frac{1}{3} \qquad \text{or } \theta = 18.43°$$

$$\therefore \quad C = 0.36$$

To find s: It is to be determined from Fig. 12.6(b).

$$z = 400 \text{ m.}$$
$$L = 400 \times 3 = 1200 \text{ m.}$$
$$L_c = \frac{z}{0.3} = \frac{400}{0.3} = 1333.3 \text{ m}$$
$$\frac{H}{L_c} = \frac{9}{1333.33} = 0.00675$$
$$\frac{x}{L_c} = \frac{250}{1333.3} = 0.1875$$
$$\therefore \quad s = 1.0$$
$$\therefore \quad k_3 = 1 + Cs = 1 + 0.36 \times 1 = 1.36$$

Design wind speed

$$V_z = k_1 k_2 k_3 V_b$$
$$= 1.06 \times 0.93 \times 1.36 \times 39$$
$$= 52.29 \text{ m/sec.}$$

$\therefore$ Design wind pressure

$$p_z = 0.6 \times 52.29^2 = 1640 \text{ N/m}^2$$
$$p_z = 1.64 \text{ kN/m}^2 \qquad \textbf{Answer}$$

Example 12.3

Determine the design loads on the purlins of an industrial building near Visakhapatnam, given:

Class of building: General with life of 50 years.

Terrain: Category 2.

Maximum dimension: 40 m.

Width of building: 15 m.

Height at eve level: 8 m.

Topography: θ less than 3°.

Permeability: Medium

Span of truss: 15 m

Pitch: $\frac{1}{5}$

Sheeting: A.C. sheets.

Spacing of purlins: 1.35 m.

Spacing of trusses: 4 m.

Solution:

1. *D.L. Calculations*:

Weight of sheeting including laps and connectors = 170 N/m^2.

Self wt. of purlin (assumed) = 100 N/m^2.

Total *D.L.* on purlin = 170 + 100

$= 270$ N/m^2.

Spacing of purlins = 1.35 m

$\therefore$ *D.L.* on purlin = 270 × 1.35 = 364.5 N/m

2. *Live Load*:

Span of truss = 15 m

$$\text{Pitch} = \frac{1}{5} \qquad \therefore \text{ Rise } = \frac{1}{5} \times 15 = 3 \text{ m}$$

$$\therefore \tan\theta = \frac{3}{7.5} \qquad \text{or} \qquad \theta = 21.8^\circ$$

$\therefore$ Live load on purlin = 750 – (21.8 – 10) × 20 = 514 N/m^2.

Spacing of purlins = 1.35 m

$\therefore$ Live load = 514 × 1.35 = 693.9 N/m

3. *Wind Load*:

Basic wind velocity near Visakhapatnam = 50 m/sec.

$k_1 = 1.0$

k_2 for category 2, class B building with height 8 m, is 0.98

$k_3 = 1.0$

$\therefore$ Design wind speed = $V_z = 1.0 \times 0.98 \times 1.0 \times 50$

$= 49$ m/sec.

$\therefore$ Design wind pressure $p_d = 0.6 \times 49^2 = 1440$ N/m^2.

Wind Pressure Coefficients:

$$\frac{h}{w} = \frac{10}{15} = \frac{2}{3}$$

$$\text{Thus } \frac{1}{2} < \frac{h}{w} < \frac{3}{2}$$

From Table 12.3,

When wind angle 0°, for rafter slope 21.8° [wind normal to ridge]

On windward side $C_{pe} = -0.7 + \frac{1.8}{10} \times 0.5 = -0.61$

On leeward side $C_{pe} = -0.5$

When wind angle 90°, for rafter slope 21.8° [wind parallel to ridge]

On windward side $C_{pe} = -0.8$

On leeward side $C_{pe} = -0.6 - \frac{1.8}{10} \times 0.2$

$= -0.636$

Internal wind pressure coefficient:

For a building with medium permeability

$C_{pi} = \pm 0.5$

$\therefore$ Design wind pressure on windward side = $(-0.8 - 0.5)\, p_d$

$= -1.3 \times 1440 = -1872$ N/m^2

Design wind load on leeward side = (– 0.636 – 0.5) × 1440 kN/m

$= -1635.8$ N/m^2.

$\therefore$ For purlin design:

DL + LL = 364.5 + 694.9 = 1059.4 N/m.

= 1.0594 kN/m.

Wind Load = – 1872 N/m^2 = – 1872 × 1.35 N/m.

= –2527.2 N/m.

= –2.527 kN/m.

Hence purlin should be designed for

(a) LL + DL of 1.058 kN/m acting vertically downward

(b) DL of 0.365 kN/m acting vertically downward and WL of 2.527 kN/m, suction.

[*Note:* Design of purlins is as explained in chapter 7 Figs. 7.13 to 7.15. (examples: 7.9, 7.10 and 7.11)]

12.10 LOADS ON TRUSSES

The following loads per square metre of plan area may be used for the design of trusses.

Dead loads:

Roof Coverings:

1. GI sheets = 100 to 150 N/m^2

   A.C. sheets = 170 to 200 N/m^2.

2. Purlins = 100 to 120 N/m^2.

3. Self wt. of truss = $20 + 6.6L$ N/m^2

$$\text{or} \quad = 10\left(\frac{L}{3}+5\right)\frac{s}{4}\ \text{N/m}^2$$

Live Loads:

For a slope $\theta < 10°$, $LL = 750$ N/m^2.

For a slope > 10°, $LL = \frac{2}{3}[750 - (\theta - 10)20]$

[*Note:* it is $\frac{2}{3}$ rd of LL used in purlin design]

Wind load:

To be calculated as explained in Fig. 12.8. Loads on panel points: The loads calculated per plan area should be multiplied by area taken care by each panel point. Thus for truss design loads are considered to act at panel points (joints) of the truss.

12.11 ANALYSIS OF TRUSSES

Treating it as pinjointed structure, the truss is analysed for the following loads separately:

(a) For dead load
(b) For live load
(c) For wind load

For each member maximum and minimum design forces are to be found, taking tension as positive. For this the following load combinations are considered

(i) DL + LL
(ii) DL + WL

Some of the members may be under tension only for all combination of loads, some may be always under compression and some of them may be under tension for some load combinations and under compression for some other load combinations.

12.12 GROUPING OF MEMBERS

From aesthetic point and from the point of fabrication, it is not desirable to design each and every member for its design forces. Usually the following groups are made and the designs are made to maximum design force of a member in that group:

1. Top chord members
2. Bottom chord members
3. Main slings
4. Main struts
5. Other minor members.

Tubular sections, single angles or double angles are used as members of trusses. The minimum thickness of angles used is 6 mm. The following are the commonly used minimum sections in roof trusses:

Top chord:	2 ISA 7550, 6 mm.
Bottom chord:	2 ISA 7550, 6 mm.
Main sling:	2 ISA 6545, 6 mm.
Main strut:	2 ISA 6545, 6 mm.
Minor members:	ISA 6545, 6 mm or ISA 5050, 6 mm.

12.13 DESIGN OF MEMBERS

The member subjected to maximum design force in each group is designed as explained in chapters 5 and 6. If the design force is tensile for some loading and compression for some other loadings, the member may be designed for major force and checked for the other force. If the purlins are not at panel points, the top chord member is to be designed for combined bending and compression.

12.14 DESIGN OF JOINTS

Gusset plates are used at joints to connect various members at that joint. Gusset plate thickness is kept more than the maximum thickness of member meeting at the joint. Care should be taken to see

that centroid of all members meet at the required point of the joint. The connections may be bolted or welded.

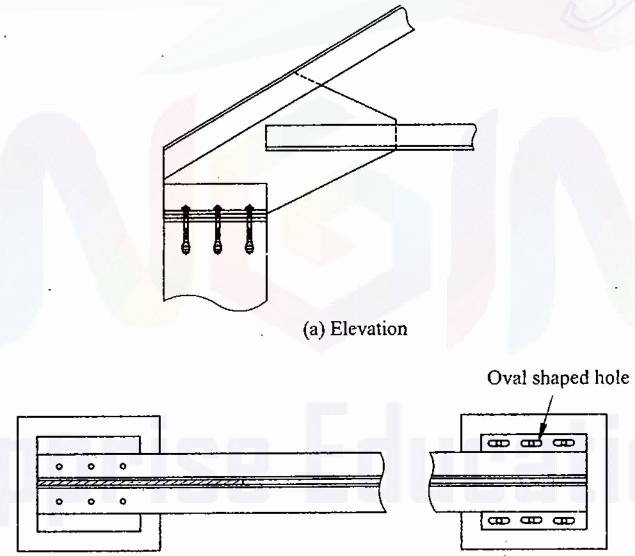
12.15 END BEARINGS

Roof trusses are supported on steel columns or concrete columns or on walls. One end is made hinged and the other sliding so that the horizontal forces on the supporting structures are avoided and at the same time the truss analysis carried as hinged-simply supported and holds good.

To achieve this a base plate is fixed to the supporting structure and another plate to the truss. To fix base plate to supporting structure anchor bolts are used. Hinged/fixed end is achieved by providing holes of hardly any tolerance for end connection, while to achieve sliding end oval shape holes with 2 to 4 mm tolerance are made in base plate of truss for anchor bolts.

Base plate is designed for bending due to end reaction while anchor bolts are designed to resist uplift force.

Figure 12.10 shows the details of end connection and Fig. 12.11 shows few typical foundation bolts.



(b) Plan of fixed/hinged end (c) Plan of sliding end

Figure 12.10 End bearing in trusses.

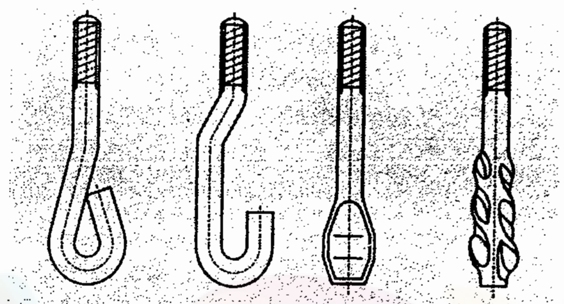


Figure 12.11 Foundation bolts.

Example 12.4

Design a truss of span 15 m, spacing 4 m to be built near Visakhapatnam, other details being same as given in example 12.3.

Solution:

1. *Selection of Configuration*:

Let a pitch of $\frac{1}{5}$ be provided.

$\therefore$ Height of truss $= \frac{1}{5} \times 15 = 3$ m

$\therefore$ Slope of top chord $= \tan^{-1}\frac{3}{7.5} = 21.8°$

If purlins are to be placed on top panel point only, panel length should be around 1.4 m so that sufficient lap can be provided when 1.65 m A.C. sheets are used.

Length of top chord $\sqrt{7.5^2 + 3^2} = 8.078$ m.

If we select 6 panels, length of panel $= \frac{8.078}{6} = 1.346$ m say 1.35

Hence Fan-Type truss shown in Fig. 12.12 is selected. [*Note:* It is not absolutely necessary to provide purlins always on panel point. When they are not on panel points, top chord members are to be designed for bending also].

2. *Loads*:

DL: As in the example 12.3,

Wt. of sheeting including laps and connections = 170 N/m^2

Wt. of purlins = 120 N/m^2

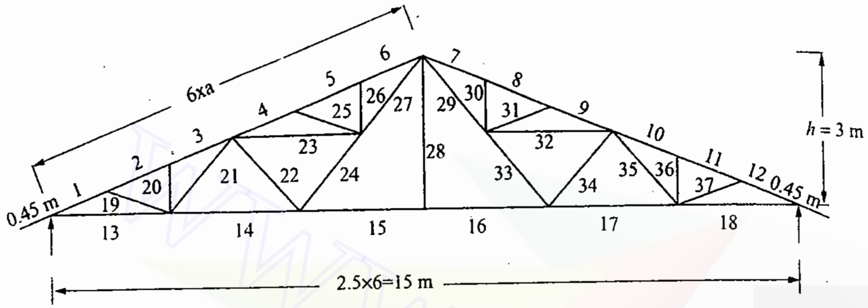


Figure 12.12

Self wt. of truss = 20 + 6.6 L

$$= 20 + 6.6 \times 15 = 119 \text{ N/m}^2, \text{ say } 120 \text{ N/m}^2$$

$\therefore$ Total dead load = 170 + 120 + 120 = 410 N/m^2

Each purlin takes care of an area = 1.35×4 m^2.

$\therefore$ Load on each intermediate panel point = $410 \times 1.35 \times 4$

= 2214 N

= 2.214 kN.

Load on shoe: Taking 450 mm roof projection load $= 410 \times \left(\frac{1.35}{2} + \frac{0.45}{2}\right) \times 4 = 1476 \text{ N} = 1.476 \text{ kN}$

Live load:

LL = 750 – (21.8 – 10) × 20 = 514 N/m^2

$\therefore$ LL on intermediate panel point = 514 × 1.35 × 4 = 2776 N = 2.776 kN

LL on shoe $= 514 \times \left(\frac{1.35}{2} + \frac{0.45}{2}\right) \times 4 = 1850 \text{ N} = 1.850 \text{ kN}$

Wind Load:

As in example 12.3,

Wind pressure on windward side = –1.872 kN/m^2

and wind pressure on leeward side = –1.636 kN/m^2.

Wind load on panel points on windward side:

(a) Intermediate panels = – 1.872 × 1.35 × 4 = – 10.109 kN

(b) At crown joint = – 5.050

(c) At shoe $= -1.872\left(\frac{1.35 + 0.450}{2}\right) \times 4$

$= -6.74$ kN

Wind load on leeward side:

(a) Intermediate panel = –1.636 × 1.35 × 4 = – 8.83 kN

(b) At crown joint = – 4.415 kN

(c) Shoe $= -1.636\left(\frac{1.35 + 0.450}{2}\right) \times 4$

$= -5.9$ kN.

3. *Analysis*:

The truss is analysed for the dead loads as shown in Fig. 12.13 and dead load forces in various members are entered in column 3 of Table 12.5. Since live load is 514 N/m^2 and dead load is 410 N/m^2, the forces in various members due to live load are found by mutiplying the values of member forces obtained for dead load with 514/410.

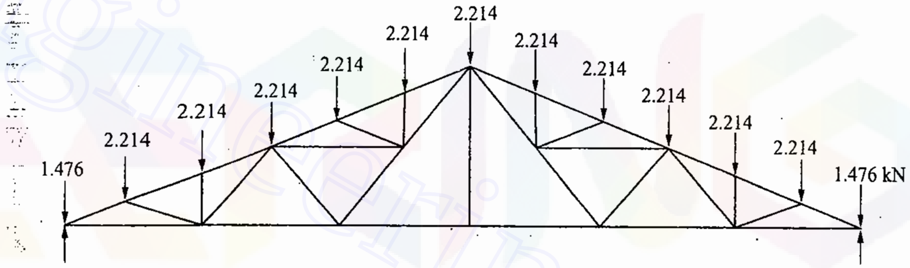


Figure 12.13

Wind load analysis is carried out for the loads as shown in Fig. 12.14 and the member forces are entered in Table 12.5.

The above analyses were carried out using software STAAD PRO 2005. They may be carried out using method of joint clubbed with method of section also.

4. *Design Forces*:

It may be observed that in a member dead loads and live load produce forces of same nature while wind load produces force of opposite nature. Hence for getting design forces the following combinations are to be considered:

Table 12.5 Member forces [Tension +Ve, compression –Ve]

Group	Member	Forces in the members in kN due to DL	LL	WL
I	1	– 32.784	– 41.105	133.438
	2	– 29.807	– 37.368	122.823
	3	– 29.807	– 37.368	126.867
	4	– 28.317	– 35.500	123.581
	5	– 25.336	– 31.763	112.966
	6	– 25.336	– 31.763	117.010
	7	– 25.336	– 31.763	112.621
	8	– 25.336	– 31.763	109.084
	9	– 28.317	– 35.500	118.360
	10	– 29.807	– 37.368	121.230
	11	– 29.807	– 37.368	117.698
	12	– 32.784	– 41.105	126.970
II	13	30.443	38.165	– 124.466
	14	24.908	31.226	– 97.244
	15	16.605	20.817	– 56.410
	16	16.605	20.817	– 56.410
	17	24.908	31.226	– 92.071
	18	30.443	38.165	– 115.845
III	27	10.807	13.548	– 53.154
	24	6.484	8.041	– 31.892
	29	10.807	13.548	– 46.420
	33	6.484	8.041	– 27.852
IV	22	– 6.484	– 8.041	31.892
	34	– 6.484	– 8.041	27.852
V	23	5.535	6.939	– 27.223
	32	5.535	6.939	– 23.774
	21	4.323	5.420	– 21.261
	35	4.323	5.420	– 18.568
VI	19	– 2.981	– 3.757	14.660
	20	– 2.214	– 2.776	10.889
	25	– 2.981	– 3.737	14.660
	26	– 2.214	– 3.737	14.660
	37	– 2.981	– 2.776	10.889
	36	– 2.214	– 3.737	12.809
	31	– 2.281	– 3.737	12.803
	30	– 2.244	– 2.813	9.510
	28	0	0	0

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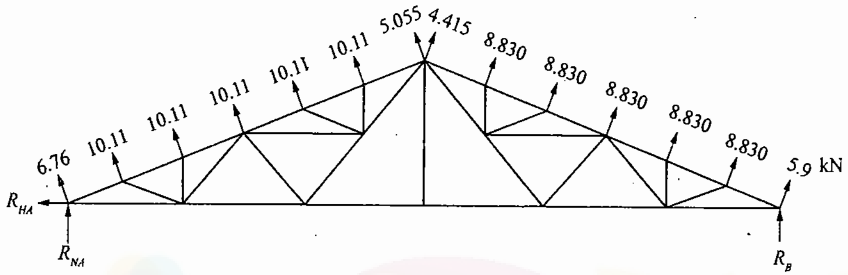

Figure 12.14

(i) DL + LL
(ii) DL + WL

From Table 4 of IS 800-2007, we find load factor is 1.5 for load case (i) whereas for load case (ii) it is 0.9 for DL and 1.5 for WL. Hence the factored force in a member is to be found for

(i) 1.5 × (Force due to DL + Force due to LL)
(ii) 1.5 × Force due to DL + 1.5 Force due to WL.

In each group combination of design forces are checked for various members and the one which gives maximum +ve and maximum –ve force is picked up.

For *group I*: (Top chord members) the design forces are

1.5 (– 32.788 – 41.105) = –110.840 kN

and 1.5 (–32.788) + 1.5 × 133.438 = 150.975 kN

For group II: (Bottom chord members)

1.5 (30.443 + 38.165) = 102.912 kN

and 1.5 × 30.443 – 1.5 × 124.466 = –141.035 kN

For group III: [Main slings]

1.5 (10.807 + 13.548) = 36.533 kN

and 1.5 × 10.807 – 1.5 × 53.154 = – 63.521 kN

For group IV:

1.5 (– 6.484 – 8.041) = – 21.788 kN

and 1.5 (– 6.484) + 1.5 × 31.892 = 38.112 kN

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For group V:

$$1.5\,(5.535 + 6.939) = 18.711 \text{ kN}$$

$$1.5 \times 5.535 - 1.5 \times 23.774 = -27.359 \text{ kN}$$

For group VI:

$$1.5\,(-2.881 - 3.737) = -9.927 \text{ kN}$$

$$1.5 \times (-2.981) + 1.5 \times 14.660 = 17.519 \text{ kN}$$

5. *Design of Members*:

A member in each group is designed for the major force and checked for minor force. The design procedure for tension is already explained in chapter 5 and the procedure for the design of compression member has been explained in chapter 6.

6. *Design of Connections*:

Using gusset plates of thickness more than the thickness of members and connections are designed using bolts or welding. Bolted connection design has been already explained in chapter 3 and welded connection design procedure has been explained in chapter 4.

7. Suitable end bearings are designed, which depend whether the truss is supported on steel column or concrete column or on masonry.
8. Drawing: Figure 12.15 shows the detailed drawing.

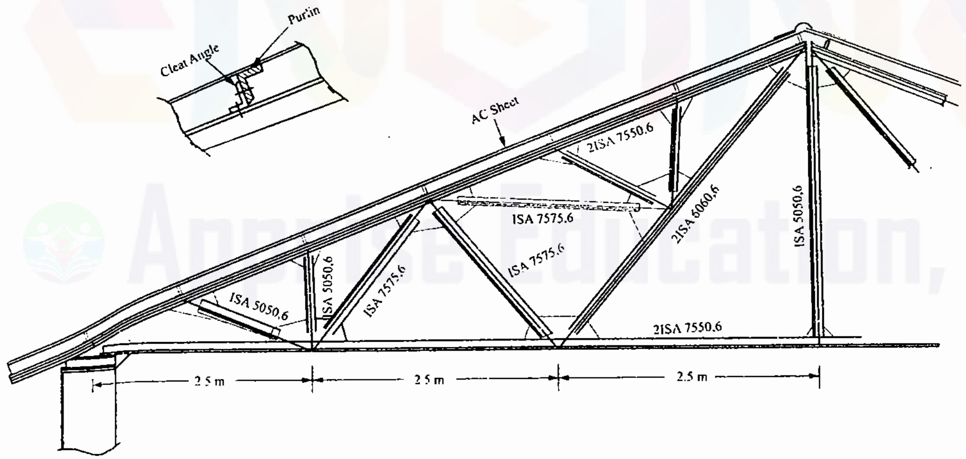


Figure 12.15 Design details of trusses.

Questions

1. Explain the following with neat sketches:
   (a) Bracing system in roof trusses
   (b) Anchorages of truss with concrete column
   (c) Connection of purlin to rafter.
2. Determine the basic wind pressure to be considered for a shed in the outskirts of Bangalore. Given:

   Structure: General purpose with probable life of 50 years.
   Terrain category: I, Building class B.
   Eye board height: 11 m.
   Topography: Plain area.
3. Determine the basic wind pressure on a pitched roof near Poona. Given:

   Structure: General purpose with probable life of 50 years.
   Terrain category: II, Building class A
   Topography: Height of hill = 350 m.
   Slope 1 in 4.
   Location of the building: 300 m from the crest of the hill on downward slope.
   Height of eye board: 10 m.
4. Determine the various loads to be considered for designing a truss near Jabalpur, for the following data given:

   Class of Building: General with life of 50 years.
   Terrain category: 2
   Size of building: 18 m × 40 m.
   Height of eye board: 12 m.
   Topography: Plan area (slope < 3°)
   Permeability: Medium
   Span of truss = 18 m.
   Pitch $\frac{1}{4}$
   Sheeting: A.C. Sheets.
   Spacing of purlins: 1.4 m.
   Spacing of trusses: 5 m.
5. The critical members of top chord, bottom chord and main sling of a truss are subjected to the forces as shown below:

Member	DL	LL	WL
Top chord	– 28.2	– 36.66	124.60
Bottom chord	26.40	34.32	– 116.40
Main sling	10.40	13.52	– 48.80

   Determine the factored forces to be considered for the design of each member.

APPENDIX

Properties of Standard Steel Sections

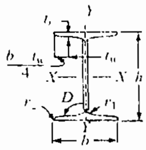
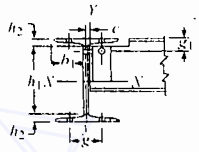


TABLE I ROLLED STEEL BEAMS

DIMENSIONS AND PROPERTIES



Designation	Weight per metre w in N	Sectional Area A cm²	Depth of section h mm	Width of Flange b mm	Thickness of Flange tf mm	Thickness of Web tw mm	Moments of inertia I_{xx} cm⁴	I_{yy} cm⁴	Radii of Gyration r_{xx} cm	r_{yy} cm	Moduli of section Z_{xx} cm³	Z_{yy} cm³	Radius at Root r_1 mm	Radius at Toe r_2 mm	Connection details h_1 mm	h_2 mm	g mm	g1 mm	Plastic Modulus of Section (Zp) cm³
ISLB 75	60	7.71	75	50	5.0	3.7	22.7	10 0	3.07	1.14	19.4	4.0	6.5	2.0	51.7	11.65	30	--	22.4
ISJB 150	70	9.01	150	50	4.6	3.0	322.1	9.2	5.98	1 01	42 9	3.7	5.0	1.5	130 4	9.8	30	45	49.6
ISLB 100	78	10.21	100	50	6.4	4.0	168	12.7	4.06	1.12	33.6	5.1	7.0	3.0	73	13.5	30	50	38.9
ISJB 175	79	10.28	175	50	4.8	3.2	479 3	9 7	6.83	0.97	54.8	3.9	5.0	1.5	155	10	30	45	64.2
ISJB 200	97	12.64	200	60	5.0	3.4	780 7	17.3	7.86	1.17	78 1	5.8	5.0	1.5	179.5	10.25	30	45	90.9
ISMB 100	113	14 60	100	75	7 2	4.0	257.5	40.8	4.2	1.67	51.5	10.9	9.0	4.5	65	17.5	35	55	58.7
ISLB 125	117	15.12	125	75	6.5	4 4	406.8	43.4	5.19	1.69	65.1	11.6	8.0	3.0	95.4	14.8	35	50	73.9
ISJB 225	126	16.28	225	80	5.0	3.7	1308.5	40.5	8.97	1.58	116.3	10.1	6.5	1.5	201 1	11.95	40	45	134.2
ISMB 125	128	16.60	125	75	7.6	4.4	449	43.7	5.2	1.62	71.8	11.7	9.0	4.5	89.2	17.9	35	55	81.8
ISLB 150	139	18.08	150	80	6.8	4.8	688.2	55.2	6.17	1.75	91.8	13.8	9.5	3.0	116.9	16.55	40	50	104.5
ISMB 150	146	19.00	150	80	7.6	4.8	726.4	52.6	6.18	1.66	96.9	13.1	9.0	4.5	113.9	18.05	40	55	110.5
ISLB 175	164	21.30	175	90	6.9	5.1	1096.2	79.6	7.17	1.93	125.3	17.7	9.5	3.0	141 6	16.7	50	50	143.3
ISWB 150	167	21.87	150	100	7·0	5·4	839·1	94·8	6.22	2·09	111·9	19·0	8.00	4·0	116.6	16.7	55	55	126.9
ISMB 175	191	24.62	175	90	8.6	5.5	1272	85.0	7.19	1.66	145.4	18.9	10.0	5.0	134.5	20.25	50	55	166.1
ISLB 200	194	25.27	200	100	7.3	5.4	1696.6	115.4	8.19	2.13	169.7	23.1	9.5	3.0	165 7	17.15	55	50	184.3
ISWB 175	217	28.11	175	125	7·4	5·8	1509·4	188·6	7.33	2·59	172.5	30·2	8·0	4·0	139.5	17.75	65	55	194.2
ISLB 225	231	29.92	225	100	8.6	5.8	2501.9	112.7	9.15	1.94	222.4	22.5	12.0	6.0	180 3	22.35	55	55	254.7
ISMB 200	249	32.33	200	100	10.8	5.7	2235.4	150.0	8.32	2 15	223.5	30.0	11.0	5.5	152.7	23.65	55	60	253.9
ISHB 150	266	34.48	150	150	9·0	5.4	1455·6	431·7	6.5	3·54	194·1	57·6	8·0	4·0	112	19	90	55	215.6
ISLB 250	274	35.53	250	125	8.2	6.1	3717.8	193 4	10.23	2.33	297.4	30.9	13.0	6.5	202.6	23.7	65	60	338.7
ISWB 200	283	36.71	200	140	9·0	6·1	2624·5	328·8	8.46	2·99	262·5	47·0	9·0	4·5	156.8	20.6	80	55	293.8
ISHB 150	300	38.98	150	150	9·0	8·4	1540·0	460·3	6.29	3·44	205·3	60·2	8·0	4·0	112	19	90	55	232.5
ISMB 225	306	39.72	225	110	11.8	6.5	3441.8	218.3	9.31	2.34	305.9	39.7	12.0	6.0	173.3	25.85	60	60	348.3
ISLB 275	324	42.02	275	140	8.8	6.4	5375.3	287.0	11.31	2.61	392.4	41.0	14.0	7.0	223.7	25.65	80	60	443.6
ISWB 225	333	43.24	225	150	9·9	6·4	3920·5	448·6	9.52	3·22	348·5	59·8	9·0	4·5	181.4	21.8	90	55	389.5
ISHB 150	339	44.08	150	150	9·0	11·8	1535·6	494·9	6.09	3·35	218·1	63·2	8.00	4·0	112	19	90	55	251.6
ISHB 200	366	47.54	200	200	9·0	6·1	3608.4	967.1	8.71	4·51	360·8	96·7	9·0	4·5	158.4	20.8	140.1	55	397.7
ISMB 250	365	47.55	250	125	12.5	6.9	5131.6	334.5	10.39	2.65	410.5	53.5	13.0	6.5	194.1	27.95	65	65	468.7

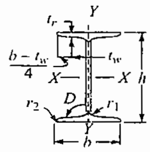


TABLE I ROLLED STEEL BEAMS

DIMENSIONS AND PROPERTIES

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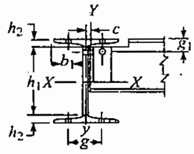

Designation	Weight per metre w in N	Sectional Area a cm²	Depth of section h mm	Width of Flange b mm	Thickness of Flange tf mm	Thickness of Web tw mm	Moments of inertia I_{xx} cm⁴	I_{yy} cm⁴	Radii of Gyration r_{xx} cm	r_{yy} cm	Moduli of section Z_{xx} cm³	Z_{yy} cm³	Radius at Root r_1 mm	Radius at Toe r_2 mm	Connection details h_1 mm	h_2 mm	g mm	g1 mm	Plastic Modulus of Section (Zp) cm³
ISLB 300	370	48.08	300	150	9.4	6.7	7332.9	376.2	12.35	2.8	488.9	50.2	15.0	7.5	245.1	27.45	90	60	554.3
ISHB 200	392	50.94	200	200	9·0	7·8	3721·8	994·6	6.55	4.42	372.2	98·6	9.00	4.50	158.4	20.8	140.1	55	414.2
ISWB 250	401	52.05	250	200	9·0	6·7	5943.1	875.5	10.69	4·06	475·4	85·7	10·0	5·0	203.8	23.1	140.0	60	527.7
ISLB 325	423	54.90	325	165	9.8	7.0	9847.6	510.8	13.41	3.05	607.7	61.9	16.0	8.0	266.5	29.25	100	65	687.8
ISHB 225	423	54.94	225	225	9·1	6·5	5279.5	1353·8	9.8	4·96	469·3	120·3	10·0	5·0	180.5	22.2	140	55	515.6
ISMB 300	402	56.26	300	140	12.4	7.5	8603.6	453.9	12.37	2.84	573.6	64.8	14.0	7.0	241.5	29.25	80	65	651.7
ISHB 225	459	59.66	225	225	9.1	8.6	5478·8	1396·6	9.58	4·84	487·0	123·0	10·0	5·0	160.5	22.2	140	55	542.2
ISWB 300	472	61.33	300	200	10·0	7·4	9821.6	990.1	12.66	4·02	654·8	99·0	11·0	5·5	250.1	24.95	140.1	60	731.2
ISLB 350	472	63.01	350	165	11.4	7.4	13158.3	631.9	14.45	3.17	751.9	76.6	16.0	8.0	288.3	30.85	100	65	851.7
ISHB 250	500	64.96	250	250	9·7	6·9	7736·5	1961·3	10.91	5·49	618·9	156·9	10·0	5·0	203.5	23.2	140	60	678.7
ISMB 350	514	66.71	350	140	14.2	8.1	13630.3	537.7	14.29	2.84	778.9	76.8	14.0	7.0	288	31	80	65	869.6
ISHB 250	537	69.71	250	250	9·7	8·8	7983.9	2011·7	10.7	5·37	638·7	159·7	10·0	5·0	203.5	23.2	140	60	708.4
ISLB 400	558	72.43	400	165	12.5	8.0	19305.3	716.4	16.33	3.15	965.3	86.8	16.0	6.0	336.2	31.9	100	65	1099.5
ISWB 350	558	72.5	350	200	11.40	8·0	15521·7	1175.9	14.63	4·03	887	117·6	12·0	6·0	295.5	27.25	140.1	60	995.5
ISHB 300	577	74.85	300	250	10·6	7·6	12545·2	2193·6	12.95	5·41	836·3	175·5	11.00	5.50	249.8	25.1	140	60	924.7
ISMB 400	604	78.46	400	140	16.0	8.9	20458.4	622.1	16.15	2.82	1022.9	88.9	14.0	7.0	334.4	32.8	80	70	1176.2
ISHB 300	618	80.25	300	250	10'6	9·4	12950·2	2246.7	12.7	5·29	863·3	178·4	11·0	5.50	249.8	25.1	140	60	962.2
ISLB 450	641	83.14	450	170	13.4	8.6	22536.1	853.0	18.2	3.2	1223.8	100.4	16.0	8.0	384	33	100	70	1401.4
ISWB 400	654	85.01	400	200	13·0	8·6	23426·7	1388·0	16.6	4·04	1171·3	138·8	13·0	6·5	340.5	29.75	140. 1	65	1290.2
ISHB 350	661	85.91	350	250	11·6	8·3	19 159·7	2451.4	14.93	5·34	1094.8	196·1	12·0	8.00	296	27	140	60	1213.5
ISHB 350	710	92.21	350	250	11·6	10.1	19802·8	2510·5	14.65	5·22	1131·6	199·4	12.00	6.00	296	27	140	60	1268.7
ISMB 450	710	92.27	450	150	17.4	9.4	30390.8	834.0	18.15	3.01	1350.7	111.2	15.0	7.5	379.2	35.4	90	70	1553.4
ISLB 500	736	95.5	500	180	14.1	9.2	38579	1063.9	20.1	3.34	1543.2	118.2	17.0	8.5	430.2	34.9	100	70	1773.7
ISHB 400	759	98.66	400	250	12·7	9·1	28083·5	2728·3	16.87	5·26	1404·2	218·3	14.00	7·0	340.1	29.9	140	65	1566.4
ISWB 450	779	101.15	450	200	15·4	9·2	35057·6	1706·7	18.63	4·11	1558·1	170·7	14·0	7·0	384	33	140.1	70	1760.6
ISHB 400	806	104.66	400	250	12·7	10·6	28823·5	2783·0	16.61	5·16	1444·2	221·3	14·0	7·0	340.1	29.9	140	65	1624.6
ISLB 550	847	109.97	550	190	15.0	9.9	53161.6	1335.1	21.99	3.48	1933.2	140.5	18.0	9.0	476.1	36.95	100	70	2228.2
ISMB 500	852	110.74	500	180	17.2	10.2	45218.3	1369.8	20.21	3.52	1808.7	152.2	17.0	8.5	424.1	37.95	100	75	2074.7

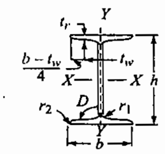

TABLE I ROLLED STEEL BEAMS

DIMENSIONS AND PROPERTIES

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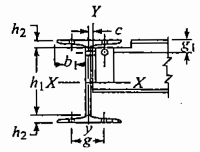


Designation	Weight per metre	Sectional Area	Depth of section	Width of Flange	Thickness of Flange	Thickness of Web	Moments of Inertia		Radii of Gyration		Moduli of section		Radius at Root	Radius at Toe	Connection details				Plastic Modulus of Section
	w	a	h	b	tf	tw	I_{xx}	I_{yy}	r_{xx}	r_{yy}	Z_{xx}	Z_{yy}	r_1	r_2	h_1	h_2	g	$g1$	(Zp)
	In N	cm^2	mm	mm	mm	mm	cm^4	cm^4	cm	cm	cm^3	cm^3	mm	mm	mm	mm	mm	mm	cm^3
ISHB 450	859	111.14	450	250	13·7	9·8	39210·8	2985·2	18.78	5·18	1742.7	238·8	15·0	7.50	386.2	31.9	140	65	1955.2
ISHB 450	907	111.89	450	250	13·7	11·3	40349·9	3045·0	18.5	5.08	1793·3	242·1	15·0	7.50	386.2	31.9	140	65	2030.9
ISWB 500	934	121.22	500	250	14·7	9·9	52290·9	2987·8	20.77	4·96	2091·6	239·0	15.00	7·5	431	34.5	140	70	2351.4
ISLB 600	976	126.69	600	210	15.5	10.5	72867.6	1821.9	23.98	3.79	2428.9	173.5	20.0	10.0	520.2	39.9	140.1	75	2798.6
ISMB 550	1017	132.11	550	190	19·3	11·2	64893.6	1833·8	22.16	3·73	2359·8	193·0	18·0	9.00	467.5	41.25	100	75	2712.6
ISWB 550	1104	143.34	550	250	17·6	10·5	74908·1	3740.6	22.86	5·11	2723·9	299·2	16·0	8·0	473.4	38.3	140	75	3066.3
ISMB 600	1210	156.21	600	210	20·8	12·0	91813·0	2651·0	24.24	4·11	3060·4	252·5	20·0	10·0	509.7	45.15	140.1	80	3510.7
ISWB 600	1312	170.38	600	250	21·3	11·2	106198·5	4702·5	24.97	5·25	3540	376·2	17·0	8·5	514.2	42.9	140	80	3986.7
ISWB 600	1423	164.86	600	250	23.6	11·8	115 626.6	5298·3	25.01	5·35	3854·2	423·9	18·0	9·0	507.9	46.05	140	80	4341.6

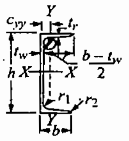
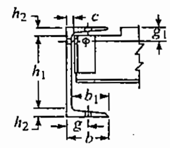


TABLE II ROLLED STEEL CHANNELS

DIMENSIONS AND PROPERTIES



Designation	Weight per metre	Sectional Area	Depth of section	Width of Flange	Thickness of Flange	Thickness of Web	Centre of Gravity	Moments of Inertia		Radii of Gyration		Moduli of section		Radius at Root	Radius at Toe	Connection details				Plastic Modulus of Section
	w	a	h	b	tf	tw	Cyy	Ixx	Iyy	rxx	ryy	Zxx	Zyy	r_1	r_2	h_1	h_2	g	g1 (Min)	(Zp)
	kg	cm^2	mm	mm	mm	mm	cm	cm^4	cm^4	cm	cm	cm^3	cm^3	mm	mm	mm	mm	mm	mm	cm^3
ISLC 75	56	7.26	75	40	6.0	3.7	1.35	66.1	11·5	3.02	1.26	17·6	4.3	6.0	2.0	50.4	12.3	21	–	20.6
ISJC 100	57	7.41	100	45	5.1	3.0	1.40	123.8	14.9	4.09	1.42	24·8	4.6	6.0	2.0	77	11.5	25	50	28.4
ISMC 75	67	8.67	75	40	7.3	4.4	1.31	76.0	12·6	2·96	1.21	20·3	4.7	8.5	4.5	41.4	16.8	21	–	24.2
ISLC 100	77	10.02	100	50	6.4	4.0	1.62	164.7	24.8	4·08	1.57	32·9	7.3	6.0	2.0	74.3	12.8	28	50	38.1
ISJC 125	77	10.07	125	50	6.6	3.0	1.64	270.0	25.7	5·18	1.60	43·2	7.6	6.0	2.5	98.9	13.1	28	50	49.1
ISMC 100	90	11.70	100	50	7.5	4.7	1.53	187.7	25·9	4·00	1.49	37·3	7.5	9.0	4.5	64.0	18	28	50	43.8
ISJC 150	97	12.65	150	55	6.9	3.6	1.66	471.1	37.9	6·10	1.73	62·8	9.9	7.0	3.0	121.2	14.4	30	50	72.0
ISLC 125	105	13.67	125	65	6.6	4.4	2.04	356.8	57·2	5.11	2.05	57.1	12.8	7.0	2.5	96.6	14.2	35	50	65.4
ISJC 175	110	14.24	175	60	6.9	3.6	1.75	719.9	50.5	7·11	1.88	82·3	11.9	7.0	3.0	146.1	14.5	35	50	94.2
ISMC 125	125	16.19	125	65	8.1	5.0	1.94	416.4	59·9	5.07	1.92	66·6	13.1	9.5	5.0	85.4	19.8	35	55	77.2
ISJC 200	136	17.77	200	70	7.1	4.1	1.97	1161.2	84.2	8.08	2.18	116.1	16.7	8.0	3.5	168.5	15.8	40	50	133.1
ISLC 150	141	18.36	150	75	7.8	4.8	2.38	697.2	103·2	6·16	2.37	93.0	20.2	8.0	3.5	117.0	16.5	40	50	106.2
ISMC 150	161	20.88	150	75	9.0	5.4	2.22	779.4	102·3	6·11	2.21	103·9	19.4	10.0	5.0	106.7	21.7	40	55	119.8
ISLC 175	173	22.40	175	75	9.5	5.1	2.40	1148.4	126·5	7·16	2.38	131·3	24.0	8.0	4.0	138.6	18.2	40	55	150.4
ISMC 175	187	24.38	175	75	10.2	5.7	2.20	1223.3	121·0	7·08	2.23	139·8	22.8	10.5	5.5	128.4	23.3	40	55	161.6
ISLC 200	202	26.22	200	75	10.8	5.5	2.35	1725.5	146·9	8.11	2.37	172.6	28.5	8.5	4.5	160.0	20	40	55	198.8
ISMC 200	217	28.21	200	75	11.4	6.1	2.17	1819.3	140.4	8·08	2.23	181·9	26.3	11.0	5.5	150.2	24.9	40	60	211.3
ISLC 225	235	30.53	225	90	10.2	5.8	2.46	2547.9	209·5	9.14	2.62	228·5	32.0	11.0	5.5	175.9	24.5	50	60	260.1
ISMC 225	254	33.01	225	80	12.4	8.4	2.30	2694.6	187·2	9·03	2.38	239·5	32.8	12.0	6.0	170.9	27.1	45	60	277.9
ISLC 250	275	35.65	250	100	10.7	6.1	2.70	3687.9	2984.0	10·17	2.89	295·0	40.9	11.0	5.5	198.6	25.5	60	60	338.1

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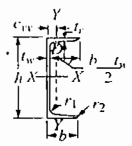
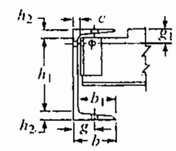


TABLE II ROLLED STEEL CHANNELS

DIMENSIONS AND PROPERTIES



Designation	Weight per metre	Sectional Area	Depth of section	Width of Flange	Thickness of Flange	Thickness of Web	Centre of Gravity	Moments of inertia		Radii of Gyration		Moduli of section		Radius at Root	Radius at Toe	Connection details				Plastic Modulus of Section
	w	a	h	b	tf	tw	Cyy	Ixx	Iyy	rxx	ryy	Zxx	Zyy	r_1	r_2	h_1	h_2	g	g1 (Mm)	(Zp)
	kg	cm^2	mm	mm	mm	mm	cm	cm^4	cm^4	cm	cm	cm^3	cm^3	mm	mm	mm	mm	mm	mm	cm^3
ISMC 250	298	38.67	250	80	14.1	7.1	2.30	3816.8	219·1	9.94	2.61	305·3	38.4	12.0	6.0	192.5	28.7	45	65	356.7
ISLC 300	325	42.11	300	100	11.6	6.7	2.55	6047.9	346·0	11·98	2.87	403·2	46.4	12.0	6.0	245.4	27.3	60	60	466.7
ISMC 300	351	45.64	300	90	13.6	7.6	2.36	6362.6	310·8	11.81	2.83	424·2	46.8	13.0	6.5	240.7	29.6	50	65	496.8
ISLC 350	381	49.47	350	100	12.5	7.4	2.41	9312.6	394.6	13.72	2.82	532·1	52.0	13.0	6.0	291.9	29.1	60	65	623.0
ISMC 350	413	53.66	350	100	13.5	8.1	2.44	10008.0	430.6	13.66	2.83	571·9	57.0	14.0	7.0	288.1	30.9	60	65	672.2
ISLC 400	448	58.25	400	100	14.0	8.0	2.36	13989.5	460.4	15.50	2.81	699·5	60.2	14.0	7.0	337.1	31.4	60	65	825.0
ISMC 400	489	62.93	400	100	15.3	8.6	2.42	15082.8	504·8	15.48	2·83	754·1	66.6	15.0	7.5	332.8	33.6	60	70	891.0

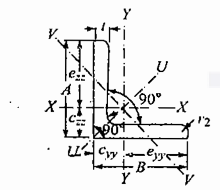
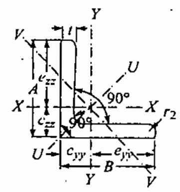


TABLE III ROLLED STEEL EQUAL ANGLES

DIMENSIONS AND PROPERTIES



Designation	Size	Thickness	Sectional Area	Weight per metre	Centre of Gravity	Distance of Extreme Fibre	Moments of Inertia			Radii of Gyration			Moduli of section	Radius at Root	Radius at Toe	Product of Inertia
	A X B	t	a	w	$C_{xx}=C_{yy}$	$e_{xx}=e_{yy}$	$I_{xx}=I_{yy}$	I_{uu}	I_{vv}	$r_{xx}=r_{yy}$	r_{uu}	r_{vv}	$Z_{xx}=Z_{yy}$	r_1	r_2	I_{xy}
	mm mm	mm	cm^2	kg	cm	cm	cm^4	cm^4	cm^4	cm	cm	cm	cm^3	mm	mm	cm^4
ISA 5050	50 X 50	5.0	4.79	37	1.41	3.59	11.0	17.6	4.5	1.52	1.92	0.97	3.10	6.00	3.00	6.50
		6.0	5.68	44	1.45	3.55	12.9	20.6	5.3	1.51	1.90	0.96	3.60			7.60
ISA 5555	55 X 55	5.0	5.27	40	1.53	3.97	14.7	23.5	5.9	1.67	2.11	1.06	3.70	6.50	4.00	8.60
		6.0	6.26	48	1.57	3.93	17.3	27.5	7.0	1.66	2.10	1.06	4.40			10.10
		8.0	8.18	63	1.65	3.85	22.0	34.9	9.1	1.64	2.07	1.06	5.70			12.80
		10.0	10.02	77	1.72	3.78	26.3	41.5	11.2	1.62	2.03	1.06	7.00			15.10
ISA 6060	60 X 60	5.0	5.75	44	1.65	4.35	19.2	30.6	7.7	1.82	2.31	1.16	4.40	6.50	4.50	11.30
		6.0	6.84	53	1.69	4.31	22.6	36.0	9.1	1.82	2.29	1.15	5.20			13.30
		8.0	8.96	69	1.77	4.23	29.0	46.0	11.9	1.80	2.27	1.15	6.80			16.90
		10.0	11.00	84	1.85	4.15	34.8	54.9	14.6	1.78	2.23	1.15	8.40			20.10
ISA 6565	65 X 65	5.0	6.25	48	1.77	4.73	24.7	39.4	9.9	1.99	2.51	1.26	5.20	6.50	4.50	14.50
		6.0	7.44	57	1.81	4.69	29.1	46.5	11.7	1.98	2.50	1.26	6.20			17.20
		8.0	9.76	76	1.89	4.61	37.4	59.5	15.3	1.96	2.47	1.25	8.10			22.00
		10.0	12.00	92	1.97	4.53	45.0	71.3	18.8	1.94	2.44	1.25	9.90			26.20
ISA 7070	70 X 70	5.0	6.77	52	1.89	5.11	31.1	49.8	12.5	2.15	2.71	1.38	6.1	7.00	4.50	18.4
		8.0	8.06	62	1.94	5.06	36.8	58.8	14.8	2.14	2.7	1.36	7.3			21.7
		8.0	10.58	81	2.02	4.98	47.4	75.5	19.3	2.12	2.87	1.35	9.5			27.9
		10.0	13.02	100	2.1	4.9	57.2	90.7	23.7	2.1	2.84	1.35	11.7			33.3
ISA 7575	75 X 75	5.0	7.27	56	2.02	5.48	38.7	61.9	15.5	2.31	2.92	1.46	7.1	7.00	4.50	22.8
		8.0	8.66	67	2.06	5.44	45.7	73.1	18.4	2.3	2.91	1.46	8.4			27
		8.0	11.38	87	2.14	5.36	59	94.1	24	2.28	2.88	1.45	11			34.8
		10.0	14.02	108	2.22	5.28	71.4	113.3	29.4	2.26	2.84	1.45	13.5			41.7
ISA 8080	80 X 80	6.0	9.29	72	2.18	5.82	56	89.6	22.5	2.46	3.11	1.56	9.6	8.00	4.50	33
		8.0	12.21	94	2.27	5.73	72.5	115.6	29.4	2.44	3.08	1.55	12.6			42.7
		10.0	15.05	116	2.34	5.66	87.7	139.5	36	2.41	3.04	1.55	15.5			51.4
		12.0	17.81	137	2.42	5.58	101.9	161.4	42.4	2.39	3.01	1.54	18.3			59.2

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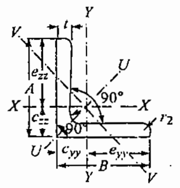
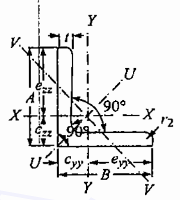


TABLE III ROLLED STEEL EQUAL ANGLES

DIMENSIONS AND PROPERTIES

(Continued)



Designation	Size	Thickness	Sectional Area	Weight per metre	Centre of Gravity	Distance of Extreme Fibre	Moments of Inertia			Radii of Gyration			Moduli of section	Radius at Root	Radius at Toe	Product of Inertia
	A X B	t	a	w	$C_{xx}=C_{yy}$	$e_{xx}=e_{yy}$	$I_{xx}=I_{yy}$	I_{uu}	I_{vv}	$r_{xx}=r_{yy}$	r_{uu}	r_{vv}	$Z_{xx}=Z_{yy}$	r_1	r_2	I_{xy}
	mm mm	mm	cm^2	kg	cm	cm	cm^4	cm^4	cm^4	cm	cm	cm	cm^3	mm	mm	cm^4
ISA 9090	90 X 90	6.0	10.47	80	2.42	6.58	80.1	128.1	32	2.77	3.5	1.75	12.2	8.50	5.50	47.2
		8.0	13.79	106	2.51	6.49	104.2	166.4	42	2.75	3.47	1.75	16			61.5
		10.0	17.03	131	2.59	6.41	126.7	201.9	51.6	2.73	3.44	1.74	19.8			74.5
		12.0	20.19	155	2.66	6.34	147.9	234.9	60.9	2.71	3.41	1.74	23.3			86.5
ISA 100100	100 X 100	6.0	11.67	90	2.67	7.33	111.3	178.1	44.5	3.09	3.91	1.95	15.2	8.50	5.50	65.7
		8.0	15.39	119	2.76	7.24	145.1	231.8	58.4	3.07	3.88	1.95	20			85.8
		10.0	19.03	146	2.84	7.16	177	282.2	71.8	3.05	3.85	1.94	24.7			104.4
		12.0	22.59	174	2.92	7.08	207	329.3	84.7	3.03	3.82	1.94	29.2			121.6
ISA 110110	110 X 110	8.0	17.02	131	3	8	195	311.7	78.2	3.38	4.28	2.14	24.4	10.00	6.00	115.1
		10.0	21.06	162	3.08	7.92	238.4	380.5	96.3	3.36	4.25	2.14	30.1			140.6
		12.0	25.02	192	3.16	7.84	279.6	445.3	113.8	3.34	4.22	2.13	35.7			164.5
		15.0	30.81	237	3.27	7.73	337.4	535.4	139.3	3.31	4.17	2.13	43.7			197
ISA 130130	130 X 130	8.0	20.22	156	3.5	9.5	328.3	525.1	131.4	4.03	5.1	2.55	34.5	10.00	6.00	194.2
		10.0	25.06	193	3.58	9.42	402.7	643.4	162.1	4.01	5.07	2.54	42.7			238.3
		12.0	29.82	230	3.66	9.34	473.8	755.9	191.8	3.99	5.03	2.54	50.7			279.9
		15.0	36.81	284	3.78	9.22	574.6	914.2	235	3.95	4.98	2.53	62.3			337.8
ISA 150150	150 X 150	10.0	29.03	224	4.06	10.94	622.4	995.4	249.4	4.63	5.86	2.93	56.9	12.00	8.00	368.2
		12.0	34.59	267	4.14	10.86	735.4	1174.8	296	4.61	5.83	2.93	67.7			435
		15.0	42.78	327	4.26	10.74	896.8	1429.7	363.8	4.58	5.78	2.92	83.5			529.1
		18.0	50.79	391	4.38	10.62	1048.9	1668.2	429.5	4.54	5.73	2.91	98.7			616
ISA 200200	200 X 200	12.0	46.61	359	5.36	14.64	1788.9	2862	715.9	6.2	7.84	3.92	122.2	15.00	10.00	1058.9
		15.0	57.8	445	5.49	14.51	2197.7	3511.8	883.7	6.17	7.79	3.91	151.4			1301.2
		18.0	68.81	530	5.61	14.39	2588.7	4130.8	1046.5	6.13	7.75	3.9	179.9			1530.5
		25.0	93.8	722	5.88	14.12	3436.3	5460.9	1411.6	6.05	7.63	3.88	243.3			2015.7

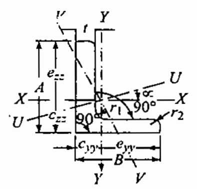
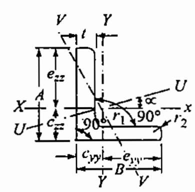


TABLE IV ROLLED STEEL UNEQUAL ANGLES

DIMENSIONS AND PROPERTIES



Designation	Size	Thickness	Sectional Area	Weight per metre	Centre of Gravity		Distance of Extreme Fibre		Moments of Inertia				Radii of Gyration				Moduli of section		tan α	Radius at Root	Radius at Toe	Product of Inertia
	A X B	t	a	w	C_{xx}	C_{yy}	e_{xx}	e_{yy}	I_{xx}	I_{yy}	I_{uu}	I_{vv}	r_{xx}	r_{yy}	r_{uu}	r_{vv}	Z_{xx}	Z_{yy}	r_1	r_1	r_2	I_{xy}
	mm mm	mm	cm^2	kg	cm	cm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^3	cm^3		mm	mm	cm^4
ISA 5030	50 X 30	5.0	3.78	29	1.72	0.74	3.28	2.26	9.3	2.5	10.3	1.5	1.57	0.81	1.65	0.63	2.8	1.1	0.35			2.7
		6.0	4.47	34	1.76	0.78	3.24	2.22	10.9	2.9	11.9	1.8	1.56	0.8	1.64	0.63	3.4	1.3	0.35			3.1
ISA 6040	60 X 40	5.0	4.76	36	1.95	0.96	4.05	3.04	16.9	6.0	19.5	3.4	1.89	1.12	2.02	0.85	4.2	2.0	0.44	6.00	4.0	5.8
		6.0	5.65	43	1.99	1.00	4.01	3.00	19.9	7.0	22.8	40	1.88	1.11	2.01	0.85	5.0	2.3	0.43			6.8
		8.0	7.37	57	2.07	1.08	3.93	2.92	25.4	8.0	29.0	5.2	1.86	1.10	1.98	0.81	6.5	3.0	0.42			8.5
ISA 6545	65 X 45	5.0	5.26	40	2.07	1.08	4.43	3.42	22.1	8.6	25.9	4.8	2.05	1.28	2.22	0.96	5.0	2.5	0.47	6.00	4.0	8.0
		6.0	6.25	48	2.11	1.12	4.39	3.38	26.0	10.1	30.4	5.7	2.04	1.27	2.21	0.95	5.9	3.0	0.47			9.4
		8.0	8.17	63	2.19	1.20	4.31	3.3	33.2	12.8	8.7	7.4	2.02	1.25	2.18	0.95	7.7	3.9	0.46			11.8
ISA 7045	70 X 45	5.0	5.52	42	2.27	1.04	4.73	3.46	27.2	8.8	30.9	5.1	2.22	1.26	2.36	0.96	5.7	2.5	0.41	6.50	4.0	8.9
		6.0	6.56	51	2.32	1.09	4.68	3.41	32.0	10.3	36.3	6.0	2.21	1.25	2.35	0.96	6.8	3.0	0.41			10.5
		8.0	8.58	66	2.40	1.18	4.6	3.34	41.0	13.1	18.3	7.6	2.19	1.24	2.32	0.95	8.9	3.9	0.40			13.2
		10.0	10.52	81	2.46	1.24	4.52	3.28	49.3	15.6	55.4	9.5	2.16	1.22	2.29	0.95	10.9	4.6	0.39			15.5
ISA 7550	75 X 50	5.0	6.02	46	2.39	1.16	5.11	3.84	34.1	12.2	39.4	69	2.36	1.42	2.56	1.07	8.7	3.2	0.44	6.50	4.0	11.6
		6.0	7.16	57	2.44	1.2	5.06	3.80	40.3	14.3	46.4	8.2	2.37	1.41	2.55	1.07	8.0	3.8	0.44			13.9
		8.0	9.36	73	2.52	1.28	4.98	3.72	51.8	18.3	59.4	10.6	2.35	1.40	2.52	1.06	10.4	4.9	0.43			17.7
		10.0	11.52	88	2.60	1.36	4.90	3.64	62.3	21.8	71.2	12.9	2.33	1.38	2.49	1.06	12.7	6	0.42			20.9
ISA 8050	60 X 50	5.0	6.27	47	2.60	1.12	5.4	3.88	40.6	12.2	45.7	7.2	2.55	1.4	2.70	1.07	7.5	3.2	0.39	7.00	4.5	12.9
		6.0	7.46	58	2.64	1.16	5.36	3.84	48.0	14.4	53.9	8.5	2.54	1.39	2.69	1.07	9.0	3.8	0.39			15.2
		8.0	9.78	76	2.73	1.24	5.27	3.76	61.9	18.5	69.3	11.0	2.52	1.37	2.66	1.06	11.7	4.9	0.38			19.3
		10.0	12.02	92	2.91	1.32	5.19	3.68	74.7	22.1	83.3	13.5	2.49	1.36	2.63	1.06	14.4	6.0	0.38			22.9
ISA 9060	90 X 80	6.0	8.65	67	2.87	1.39	6.13	4.61	70.6	25.2	81.5	14.3	2.86	1.71	3.07	1.28	11.5	5.5	0.44	7.50	5.00	24.50
		8.0	11.37	87	2.96	1.48	6.04	4.52	91.5	32.4	105.3	18.6	2.84	1.69	3.04	1.28	15.1	7.2	0.44			31.50
		10.0	14.01	108	3.04	1.55	5.96	4.45	110.9	39.1	127.3	22.8	2.81	1.67	3.01	1.27	18.6	8.8	0.43			37.80
		12.0	16.57	128	3.12	1.63	5.88	4.37	129.1	45.2	147.5	28.8	2.79	1.65	2.98	1.27	22.0	10.3	0.42			43.30

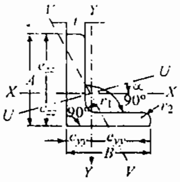
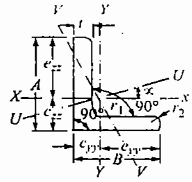


TABLE IV ROLLED STEEL UNEQUAL ANGLES

DIMENSIONS AND PROPERTIES

(Continued)



Designation	Size	Thickness	Sectional Area	Weight per metre	Centre of Gravity		Distance of Extreme Fibre		Moments of Inertia				Radii of Gyration				Moduli of section		tan α	Radius at Root	Radius at Toe	Product of Inertia
	A X B	t	a	w	C_{xx}	C_{yy}	e_{xx}	e_{yy}	I_{xx}	I_{yy}	I_{uu}	I_{vv}	r_{xx}	r_{yy}	r_{uu}	r_{vv}	Z_{xx}	Z_{yy}	r_t	r_1	r_2	I_{xy}
	mm mm	mm	cm^2	kg	cm	cm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^3	cm^3		mm	mm	cm^4
ISA 10065	100 X 65	6.0	9.55	74	3.19	1.47	6.81	5.03	96.7	32.4	110.6	18.6	3.18	1.84	3.40	1.39	14.2	6.4	0.42	8.00	5.50	32.50
		8.0	12.57	97	3.28	1.55	6.72	4.93	125.9	41.9	143.6	24.2	3.16	1.83	3.38	1.39	18.7	8.5	0.42			42.00
		10.0	15.51	120	3.37	1.63	6.63	4.87	153.2	50.7	174.2	29.7	3.14	1.81	3.35	1.38	23.1	10.4	0.41			50.70
ISA 100075	100 X 75	6.0	10.14	78	3.01	1.78	6.99	5.72	100.9	48.7	124.0	25.6	3.15	2.19	3.50	1.59	14.4	8.5	0.55	8.50	6.00	41.00
		8.0	13.36	103	3.10	1.87	6.90	5.63	131.6	63.3	161.3	33.6	3.14	2.18	3.48	1.58	19.1	11.2	0.55			53.4
		10.0	16.5	128	3.19	1.95	6.81	5.55	160.4	76.9	196.1	41.2	3.12	2.16	3.45	1.58	23.6	13.8	0.55			64.7
		12.0	19.56	151	3.27	2.03	6.73	5.47	187.5	89.5	228.4	48.6	3.10	2.14	3.42	1.58	27.9	16.3	0.54			74.9
ISA 12575	125 X 75	8.0	11.66	90	4.05	1.59	8.45	5.91	187.8	51.6	208.9	30.5	4.01	2.10	4.23	1.62	22.2	8.7	0.37	9.00	6.00	56.7
		8.0	15.38	119	4.15	1.68	8.35	5.82	245.5	67.2	272.8	40.0	4	2.09	4.21	1.61	29.4	11.5	0.36			74
		10.0	19.02	146	4.24	1.76	8.26	5.74	300.3	81.6	332.9	49.1	3.97	2.07	4.18	1.61	36.3	14.2	0.36			89.9
ISA 12595	125 X 95	6.0	12.86	99	3.70	2.22	8.80	7.28	203.2	102.1	252.3	52.9	3.97	2.82	4.43	2.03	23.1	14.0	0.57	9.00	6.00	84.5
		8.0	16.98	131	3.80	2.31	8.70	7.19	266.0	133.3	329.7	69.6	3.96	2.80	4.41	2.02	30.6	18.5	0.57			110.6
		10.0	21.02	162	3.88	2.39	8.62	7.11	325.8	162.7	402.9	85.6	3.94	2.78	4.38	2.02	37.8	22.9	0.57			135
		12.0	24.98	192	3.96	2.47	8.54	7.03	382.6	190.4	472.0	101.0	3.91	2.76	4.35	2.01	44.8	27.1	0.56			157.7
ISA 15075	150 X 75	8.0	17.42	135	5.23	1.53	9.77	5.97	407.2	70.2	432.8	44.5	4.83	2.01	4.98	1.6	41.7	11.8	0.27	10.00	6.00	95.5
		10.0	21.56	166	5.32	1.61	9.68	5.89	499.1	85.3	529.8	54.6	4.81	1.99	4.96	1.59	51.6	14.5	0.26			116.2
		12.0	25.62	197	5.41	1.69	9.59	5.81	587.0	99.5	622.2	64.3	4.79	1.97	4.93	1.58	61.2	17.1	0.26			135.2
ISA 150115	150 X 115	8.0	20.58	159	4.46	2.73	10.54	8.77	465.7	238.9	581.2	123.3	4.76	3.41	5.31	2.45	44.2	27.2	0.58	11.00	7.50	195.9
		10.0	25.52	196	4.55	2.82	10.45	8.68	573.3	293.4	714.3	152.4	4.74	3.39	5.29	2.44	54.9	33.8	0.58			241
		12.0	30.38	233	4.64	2.90	10.36	8.60	676.5	345.3	841.4	180.4	4.72	3.37	5.26	2.44	65.3	40.2	0.58			283.6
		15.0	37.52	289	4.76	302	10.24	8.48	823.5	418.6	1020.9	221.2	4.69	3.34	5.22	2.43	80.4	49.4	0.57			342.8

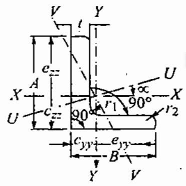
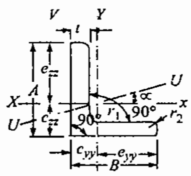


TABLE IV ROLLED STEEL UNEQUAL ANGLES

DIMENSIONS AND PROPERTIES

(Continued)



Designation	Size	Thickness	Sectional Area	Weight per metre	Centre of Gravity		Distance of Extreme Fibre		Moments of Inertia				Radii of Gyration				Moduli of section		tan α	Radius at Root	Radius at Toe	Product of Inertia
	A X B	t	a	w	C_{xx}	C_{yy}	e_{xx}	e_{yy}	I_{xx}	I_{yy}	I_{uu}	I_{vv}	r_{xx}	r_{yy}	r_{uu}	r_{vv}	Z_{xx}	Z_{yy}	r_t	r_1	r_2	I_{xy}
	mm mm	mm	cm^2	kg	cm	cm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^3	cm^3		mm	mm	cm^4
ISA 200100	200 X 100	10.0	29.03	224	6.96	2.01	13.04	7.99	1210	209.2	1286.7	132.5	6.46	2.68	6.66	2.14	92.8	28.2	0.27	12.00	8.00	284.8
		12.0	34.59	267	7.05	2.10	12.95	7.90	1431.7	246.2	1521.0	156.8	8.43	2.67	6.63	2.13	110.6	31.1	0.28			335.3
		15.0	42.78	330	7.18	2.22	12.62	7.78	1750.5	298.1	1856.7	191.9	6.40	2.64	6.59	2.12	138.5	38.3	0.26			405.4
ISA 200150	200 X 150	10.0	34	262	5.99	3.51	14.01	11.49	1377.9	669.6	1696.6	350.8	6.37	4.44	7.06	3.21	98.3	56.3	0.58	13.50	9.50	564.1
		12.0	40.56	312	6.08	3.60	13.92	11.4	1634.9	793.2	2010.6	417.2	6.35	4.42	7.04	3.21	117.4	69.6	0.56			869.1
		15.0	50.25	387	6.20	3.72	13.80	11.28	2005.6	969.9 1	2461.9	513.8	6.32	4.39	7.00	3.20	145.4	86.0	0.55			818.5
		18.0	59.76	460	6.33	3.84	13.67	11.16	2359.4	136.9	2889.5	608.9	6.28	4.36	8.95	3.19	172.5	101.9				958.1

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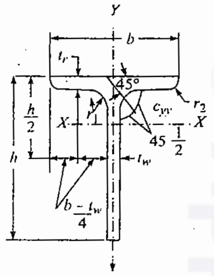
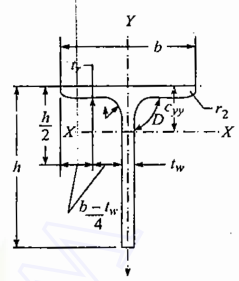


TABLE V ROLLED STEEL TEE BARS

DIMENSIONS AND PROPERTIES
(Continued)



Designation	Weight per metre w	Sectional Area a	Depth of Section h	Width of Flange b	Thickness of Flange t_f	Thickness of Web t_w	Centre of Gravity C_{xx}	Moments of Inertia I_{XX}	Moments of Inertia I_{YY}	Radii of Gyration r_{XX}	Radii of Gyration r_{YY}	Moduli of section Z_{XX}	Moduli of section Z_{YY}	Radius at Root r_1	Radius at Toe r_2
	N	cm^2	mm	mm	mm	mm	cm	cm^4	cm^4	cm	cm	cm^3	cm^3	mm	mm
ISNT 20	9	1.13	20	20	3.0	3.0	0.6	0.4	0.2	0.59	0.39	0.3	0.2	4.0	3.0
ISNT 30	14	1.75	30	30	3.0	3.0	0.8	1.4	0.6	0.89	0.57	0.6	0.4	5.0	3.5
ISNT 40	34	4.48	40	40	6.0	6.0	1.2	6.3	3.0	1.18	0.82	2.2	1.5	5.5	4.0
ISJT 75	34	4.50	75	50	4.6	3.0	2.0	24.8	4.6	2.35	1.01	4.5	1.8	5.0	1.5
ISLT 50	39	5.11	50	50	6.4	4.0	1.2	9.9	6.4	1.39	1.12	2.6	2.5	7.0	3.0
ISJT 87.5	39	5.14	87.5	50	4.8	3.2	2.5	39.0	4.8	2.75	0.97	6.2	1.9	5.0	1.5
ISNT 50	44	5.70	50	50	6.0	6.0	1.4	12.7	5.9	1.50	1.02	3.6	2.4	6.0	4.0
ISJT 100	49	6.32	100	60	5.0	3.4	2.8	63.5	8.6	3.17	1.17	8.8	2.9	5.0	1.5
ISNT 60	53	6.90	60	60	6.0	6.0	1.7	22.5	10.1	1.81	1.21	5.2	3.4	6.5	4.6
ISJT 112.5	63	8.14	112.5	80	5.0	3.7	3.0	101.6	20.2	3.53	1.58	12.3	5.1	6.5	1.5
ISLT 75	170	9.04	75	80	6.8	4.8	1.7	41.9	27.6	2.15	1.75	7.2	6.9	9.5	3.0
ISST 100	81	10.37	100	50	10.0	5.8	3.0	99.0	9.6	3.09	0.96	14.2	3.8	8.0	4.0
ISNT 80	94	12.25	80	80	8.0	8.0	2.2	71.2	32.3	2.41	1.62	12.3	8.1	8.0	5.5
ISLT 100	124	16.16	100	100	10.8	5.7	2.1	116.6	75.0	2.69	2.15	14.8	15.0	11.0	5.5
ISNT 100	147	19.10	100	100	10.0	10.0	2.8	173.8	79.9	3.02	2.05	24.1	16.0	9.0	6.0
ISHT 75	150	19.49	75	150	9.0	8.4	1.6	96.2	230.2	2.22	3.44	16.4	30.1	8.0	4.0
ISST 150	154	19.96	150	75	11.6	8.0	4.8	450.2	37.0	4.75	1.36	43.9	9.9	9.0	4.5
ISHT 100	196	25.47	100	200	9.0	7.8	1.9	193.8	497.3	2.76	4.42	24.0	49.3	9.0	4.5
ISNT 150	224	29.08	150	150	10.0	10.0	4.0	603.8	267.5	4.56	3.03	54.6	35.7	10.0	7.0
ISHT 125	267	34.85	125	250	9.7	8.8	2.4	415.4	1005.8	3.45	5.37	41.0	79.9	10.0	5.0

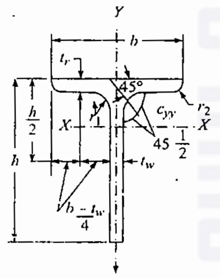
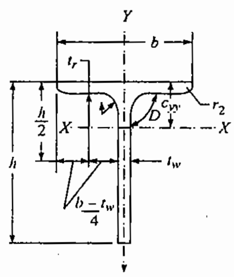


TABLE V ROLLED STEEL TEE BARS

DIMENSIONS AND PROPERTIES
(Continued)



Designation	Weight per metre w	Sectional Area a	Depth of Section h	Width of Flange b	Thickness of Flange t_f	Thickness of Web t_w	Centre of Gravity C_{xx}	Moments of Inertia I_{XX}	Moments of Inertia I_{YY}	Radii of Gyration r_{XX}	Radii of Gyration r_{YY}	Moduli of section Z_{XX}	Moduli of section Z_{YY}	Radius at Root r_1	Radius at Toe r_2
	N	cm^2	mm	mm	mm	mm	cm	cm^4	cm^4	cm	cm	cm^3	cm^3	mm	mm
ISST 200	279	36.22	200	165	12.5	8.0	4.8	1267.8	358.2	5.92	3.15	83.3	43.4	16.0	8.0
ISHT 150	288	37.42	150	250	10.6	7.6	2.7	573.7	1096.8	3.92	5.41	46.5	87.7	11.0	5.5
ISST 250	368	47.75	250	180	14.1	9.2	6.4	2774.4	532.0	7.62	3.34	149.2	59.1	17.0	8.5

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PLATES
Width: 900 1000 1100 1200 1250 1400 1500 1600 1800 2000 2200 2500
Length: 2000 2200 2500 2800 3200 3600 4000 4500 5000 5600 6300 7100 8000 9000 10000 11000 12500

Thickness: 5, 6, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56, 63

SHEET
Thickness: 0.4, 0.5, 0.63, 0.80, 0.90, 1.0, 1.12, 1.25, 1.40, 1.6, 1.8, 2.0, 2.24, 2.50, 2.8, 3.15, 3.55, 4.00
Width:600, 750, 900, 1000, 1100, 1200, 1250, 1400, 1500
Length:1800, 2000, 2200, 2500, 2800, 3200, 3600, 4000

STRIPS
Thickness: 1.6, 1.8, 2.0, 2.24, 2.50, 2.80, 3.15, 3.55, 4.00, 4.50, 5.0, 6.0, 8.0, 10.0
Width: 100, 125, 160, 200, 250, 320, 400, 500, 650, 800, 950, 1050, 1150, 1250, 1300, 1450, 1550

FLATS
Thickness: 3.0, 4.0, 5, 6, 8, 10, 12, 16, 18, 20, 25, 32, 40
Width: 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 90, 100, 110, 120, 130, 140, 150, 200, 250, 300, 400

Round Bars
Diameter: 5,,6, 8, 10, 12, 16, 20, 25, 28, 32, 36, 40, 45, 50, 56, 63, 71, 80, 90, 100, 110, 125, 140, 160, 180, 200

Square bars:
Size: 5, 6, 8, 10, 12, 16, 20, 25, 32, 40, 50, 63, 80, 100

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